

**AD-A241 911**



**91-13443**



SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

REPORT DOCUMENTATION PAGE		READ INSTRUCTIONS BEFORE COMPLETING FORM
1. REPORT NUMBER Appendix V to the O&M Manual Longview Lake	2. GOVT ACCESSION NO.	3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) Multiple-Purpose Project; Little Blue River Basin Little Blue River, Missouri Longview Lake Operation and Maintenance Manual; Appendix V Embankment Criteria and Performance Report	5. TYPE OF REPORT & PERIOD COVERED Embankment Criteria and Performance Report Sep 79 to Sep 90	
7. AUTHOR(s) Ms. Lori L. Lynch-Project Engineer Mr. Francke C. Walberg-Asst.Ch., Geotechnical Br. Mr. Robert G. Dimmitt-Ch., Dams & Foundations Sect.	6. PERFORMING ORG. REPORT NUMBER	
9. PERFORMING ORGANIZATION NAME AND ADDRESS Dams and Foundations Section (CEMRK-ED-GD) Geotechnical Branch (CEMRK-ED-G) U.S. Army Corps of Engineers, Kansas City District 601 E. 12th Street, Kansas City, MO 64106	8. CONTRACT OR GRANT NUMBER(s)	
11. CONTROLLING OFFICE NAME AND ADDRESS Civil Engineering Drafting Section (CEMRK-ED-CE) Computer Aided Drafting Branch (CEMRK-ED-C) U.S. Army Corps of Engineers, Kansas City District 601 E. 12th Street, Kansas City, MO 64106	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS PB-2b and ER1110-2-1901 (31 Dec 1981)	
14. MONITORING AGENCY NAME & ADDRESS (if different from Controlling Office)	12. REPORT DATE November 1990	
	13. NUMBER OF PAGES 333	
	15. SECURITY CLASS. (of this report) UNCLASSIFIED	
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE	
16. DISTRIBUTION STATEMENT (of this Report) Distribution in Accordance with ER 1110-2-1901, Paragraph 8, dated 31 December 1981 Distribution Statement "A"		
17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)		
18. SUPPLEMENTARY NOTES Control Determination: This report conforms to the intent of the exempt report categories as set forth in AR 335-15 and under the paragraphs 1-2y of the AR.		
19. KEY WORDS (Continue on reverse side if necessary and identify by block number)		
20. ABSTRACT (Continue on reverse side if necessary and identify by block number) This report provides the significant design, construction and operational information which can be used by engineers to (1) familiarize themselves with the project (2) reevaluate the embankment in the event unsatisfactory performance occurs and (3) provide guidance for designing comparable future projects.		

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EDITION OF 1 NOV 65 IS OBSOLETE

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SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

**Best  
Available  
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CEMRD-EP-C (CECW-EP-W/8 May 91) (10-1-7a) 1st End  
Mr. Galligan/sg/7306  
SUBJECT: Longview Lake, Missouri, Embankment Criteria and  
Performance Report

*WHL*  
*Frank*  
*GD*

DA, Missouri River Division, Corps of Engineers, P.O. Box 103,  
Downtown Station, Omaha, Nebraska 68101-0103 16 MAY 1991

FOR Commander, Kansas City District, ATTN: CEMRK-ED-GD

1. For appropriate action.
2. Please note the requested 30-day suspense.

FOR THE COMMANDER:

*JH* *DeStefano*  
JOSEPH J. GRASSO, P.E.  
Acting Director, Engineering and  
Planning Directorate



CEMRK-ED-GD (CECW-EP-W/8 May 91) 1110-2-1150a 2d End  
Ms. Lynch/sj/FTS 867-5352  
SUBJECT: Longview Lake Missouri, Embankment Criteria and  
Performance Report

DA, Kansas City District, Corps of Engineers, 700 Federal  
Building, Kansas City, Missouri 64106-2896 11 June 1991

21 JUN 1991

~~THRU Commander, Missouri River Division, ATTN: CEMRD-EP-C~~ JJC

FOR Commander, Directorate of Civil Works, ATTN: CECW-EP-W

The following are like-numbered responses to CECW-EP-W comments provided in the memorandum dated 8 May 1991 referenced in the 1st End to Longview Lake, Missouri, Embankment Criteria and Performance Report.

a. The impervious blanket was constructed on the upstream right abutment to control seepage at the abutment. The majority of the seepage was expected to occur through the highly jointed Bethany Falls limestone which exists on both abutments. An impervious blanket was required for the right abutment since the upstream bedrock was exposed. A clay blanket was not necessary for the left abutment because the upstream bedrock is covered with overburden which acts as a natural clay blanket. For a complete description of the operation and performance of the abutments see paragraphs 7-05, 9-01c, and 9-01d.

b. Excavation details and foundation treatment for the abutments are discussed in more detail in the Longview Lake Construction Foundation Report dated February 1986.

c. A banded riprap design was used on the upstream slope to minimize the amount of stone protection required. Flatter grassed slopes (1 on 50) were also used between the upstream riprap zones. The riprap extends from elevation 878 to 898 corresponding to the zone of most frequent pool level roughly the 10 year drawdown and 10 year flood and elevation 915 to 925. The information used to determine the stone protection zones is provided on plate 32.

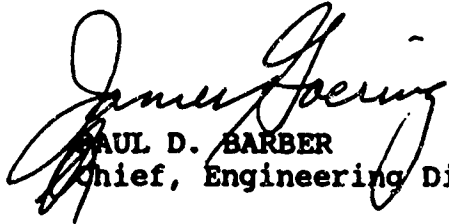
d. Reference paragraph 8-04f. Construction details for the upstream impervious blanket are provided on plate 30.

CEMRK-ED-GD

SUBJECT: Longview Lake Missouri, Embankment Criteria and  
Performance Report

e. The 10 abandoned open tube piezometers were cut off  
slightly below the ground, filled with grout and capped.

FOR THE COMMANDER:

  
PAUL D. BARBER  
Chief, Engineering Division

CECW-EP-W (CECW-EP-W/8 May 1991) 3rd End WALLACE/tf/(202)272-8890  
SUBJECT: Longview Lake Missouri Embankment Criteria and Performance  
Report

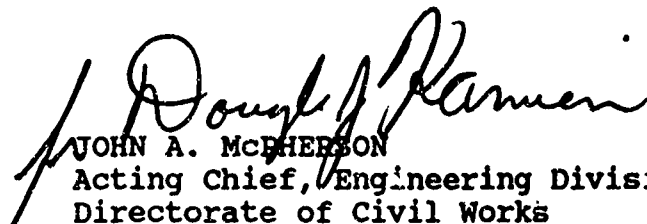
20 AUG 1991

HQ, U.S. Army Corps of Engineers, Washington, DC 20314-1000

FOR Commander, Missouri River Division, ATTN: CEMRD-EP-C

The responses provided in the preceding endorsement satisfactorily  
response to the concerns set forth in the basic letter.

FOR THE DIRECTOR OF CIVIL WORKS:

  
JOHN A. MCPHERSON  
Acting Chief, Engineering Division  
Directorate of Civil Works

10

For Kenneth Lane  
R. HARVE WIETHOP, P.E.  
Chief, Cost & Gen. Engrg. Division  
Engineering and Planning Directorate

[illegible]



DEPARTMENT OF THE ARMY  
U.S. Army Corps of Engineers  
WASHINGTON, D.C. 20314-1000

REPLY TO  
ATTENTION OF:

CECW-EP-W


08 MAY 1991

MEMORANDUM FOR Commander Missouri River Division, ATTN: CEMRD-EP-C

SUBJECT: Longview Lake, Missouri, Embankment Criteria and  
Performance Report

1. Reference memorandum CEMRK-ED-GD, 27 February 1991, subject as above with CEMRD-EP-C, 21 March 1991, first endorsement thereon.
2. The following comments on the subject report are furnished for action. Request response within 30 days as to the disposition of these comments.
3. Geotechnical concerns.
  - a. Paragraph 4-01.b. Documentation should be provided to confirm the need for the impervious blanket.
  - b. Paragraph 4-02.c. Design details respecting the abutment treatment should be provided.
  - c. Paragraph 4-04, subparagraph 4. Clarify what slope details were selected in order to minimize slope protection requirements.
  - d. Paragraph 5-02.c. Include construction details such as thickness, zones and filters for the upstream impervious blanket.
  - e. Paragraph 7.02.a. The abandonment procedures for the ten open tube piezometers should be provided.

FOR THE DIRECTOR OF CIVIL WORKS:

  
JOHN A. MOFHERSON  
Acting Chief, Engineering Division  
Directorate of Civil Works

OPERATION AND MAINTENANCE MANUAL  
LONGVIEW LAKE  
LITTLE BLUE RIVER, MISSOURI

APPENDIX V  
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

FEBRUARY 1991

DEPARTMENT OF THE ARMY  
Kansas City District, Corps of Engineers  
Kansas City, Missouri

OPERATION AND MAINTENANCE MANUAL  
LONGVIEW LAKE  
LITTLE BLUE RIVER BASIN  
LITTLE BLUE RIVER, MISSOURI

APPENDIX V  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT

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1.	March 1990. Overview of completed embankment.

CONSTRUCTION EQUIPMENT

2. 1983. Two Caterpillar D-9s loading Cat 631 scrapers in upstream valley borrow area.
3. 09 October 1981. Cat 5310 scraper.
4. 26 July 1983. Cat D-9H compacting haul road ramp with tandem sheepsfoot roller.
5. 10 July 1983. Compacting impervious fill around sump in cutoff trench with a "BARCO" gas powered compactor.
6. 06 July 1983. Spreader box mounted on a Cat D3 used for placing the 6-foot wide pervious drain.
7. June 1980. Raygo "Rascal" 410-A 5 ton vibratory roller used to compact pervious fill.
8. 07 June 1983. "Wacker" DVU4001 small vibratory plate compactor used to compact pervious fill in restricted areas.
9. 07 May 1981. Small 4 wheel "Wacker" vibratory roller.
10. 11 June 1981. Backhoe mounted vibratory plate used to compact pervious fill along the right side of the conduit.
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12. 18 July 1983. Record control sample being taken.

STAGE I EMBANKMENT CONSTRUCTION

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14. 29 May 1980. Curtain grouting in cutoff trench.
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## PHOTOGRAPHS

### Photo No.

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87	TYPICAL STABILITY ANALYSIS END OF CONSTRUCTION AND PARTIAL POOL CASE	RBL-3-1565
88	TYPICAL STABILITY ANALYSIS RAPID DRAWDOWN AND STEADY SEEPAGE	RBL-3-1566
89	EMBANKMENT STABILITY ANALYSIS SUMMARY VALLEY SECTION	RBL-3-1567
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93	EMBANKMENT MATERIAL PLACEMENT REQUIREMENTS AND QUANTITIES	RBL-3-1487
94	TOP OF DAM PROFILE PLOTS	RBL-3-1570
95	CREST SETTLEMENT MONUMENTS PLOTS	RBL-3-1571
96	CREST SETTLEMENT MONUMENTS PLOTS	RBL-3-1572
97	SETTLEMENT PLATE DATA PLOTS SPPF 91-1 AND SPPF 93-7	RBL-3-1573
98	ALIGNMENT MONUMENT HORIZONTAL PROFILE ORIGINAL SURVEY BASE LINES LINES "A" AND "B" SUMMARY	RBL-3-1574
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100	ALIGNMENT MONUMENTS HORIZONTAL AND VERTICAL DATA PLOTS MONUMENTS B-1, B-2, AND B-3	RBL-3-1576
101	PIEZOMETER PLOTS OPEN TUBE PZ PPF 84-1	RBL-3-1577
102	PIEZOMETER PLOTS OPEN TUBE PZ PPF 86-1A	RBL-3-1578
103	PIEZOMETER PLOTS OPEN TUBE PZ PPF 87-3	RBL-3-1579
104	PIEZOMETER PLOTS OPEN TUBE PZ PPF 88-2	RBL-3-1580
105	PIEZOMETER PLOTS OPEN TUBE PZ PPF 88-3	RBL-3-1581
106	PIEZOMETER PLOTS OPEN TUBE PZ PZE 90-2	RBL-3-1582
107	PIEZOMETER PLOTS OPEN TUBE PZ PPE 92-1 1 OF 2	RBL-3-1583
108	PIEZOMETER PLOTS OPEN TUBE PZ PPE 92-1 2 OF 2	RBL-3-1584
109	PIEZOMETER PLOT 2" OPEN TUBE PVC PPE 92-2 1 OF 2	RBL-3-1585

# PLATE LIST CONT'D

<u>Plate No.</u>	<u>Title</u>	<u>File No.</u>
110	PIEZOMETER PLOT 2" OPEN TUBE PVC PPE 92-2 2 OF 2	RBL-3-1586
111	PIEZOMETER PLOTS PNEUMATIC PZ PPE 94-1 1 OF 2	RBL-3-1587
112	PIEZOMETER PLOTS PNEUMATIC PZ PPE 94-1 2 OF 2	RBL-3-1588
113	PIEZOMETER PLOTS OPEN TUBE PZ PPF 99-3 2 OF 2	RBL-3-1589
114	PIEZOMETER PLOTS OPEN TUBE PZ PPF 99-4 1 OF 2	RBL-3-1590
115	PIEZOMETER PLOTS OPEN TUBE PZ PPF 99-4 2 OF 2	RBL-3-1680
116	PIEZOMETER PLOTS PNEUMATIC PZ PPE 99-8 1 OF 2	RBL-3-1681
117	PIEZOMETER PLOTS PNEUMATIC PZ PPE 99-8 2 OF 2	RBL-3-1682
118	PIEZOMETER PLOTS PNEUMATIC PZ PPE 99-11 1 OF 2	RBL-3-1683
119	PIEZOMETER PLOTS PNEUMATIC PZ PPE 99-11 2 OF 2	RBL-3-1684
120	PIEZOMETER PLOTS OPEN TUBE PZ W-6	RBL-3-1685
121	INCLINOMETER DISPLACEMENTS I-99-1	RBL-3-1697
122	INCLINOMETER DATA PLOTS I-99-1 AND I-99-2	RBL-3-1698
123	INCLINOMETER DISPLACEMENTS I-99-3	RBL-3-1699
124	INCLINOMETER DATA PLOTS I-99-3 AND I-97-1	RBL-3-1700

## SUPPLEMENT A

<u>Plate No.</u>	<u>Title</u>	<u>File No.</u>
A1	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 1 OF 16	RBL-3-1488
A2	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 2 OF 16	RBL-3-1489
A3	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 3 OF 16	RBL-3-1490
A4	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 4 OF 16	RBL-3-1491
A5	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 5 OF 16	RBL-3-1492
A6	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 6 OF 16	RBL-3-1493
A7	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 7 OF 16	RBL-3-1494
A8	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 8 OF 16	RBL-3-1495
A9	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R, R, AND GRADATION TESTS - 9 OF 16	RBL-3-1496



SUPPLEMENT A

<u>Plate No.</u>	<u>Title</u>	<u>File No.</u>
A10	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS - 10 OF 16	RBL-3-1497
A11	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS - 11 OF 16	RBL-3-1498
A12	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS - 12 OF 16	RBL-3-1499
A13	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS - 13 OF 16	RBL-3-1500
A14	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS - 14 OF 16	RBL-3-1501
A15	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS - 15 OF 16	RBL-3-1502
A16	LL 55 IMPERVIOUS STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS - 16 OF 16	RBL-3-1503
A17	LL 55 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 1 OF 3	RBL-3-1504
A18	LL 55 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 2 OF 3	RBL-3-1505
A19	LL 55 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 3 OF 3	RBL-3-1506
A20	LL 60 RANDOM STAGE I RECORD CONTROL R̄, R, AND GRADATION TESTS	RBL-3-1507
A21	LL 60 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 1 OF 3	RBL-3-1508
A22	LL 60 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 2 OF 3	RBL-3-1509
A23	LL 60 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 3 OF 3	RBL-3-1510
A24	LL 60 RANDOM STAGE II RECORD CONTROL R, R̄, AND GRADATION TESTS - 1 OF 4	RBL-3-1511
A25	LL 60 RANDOM STAGE II RECORD CONTROL R, R̄, AND GRADATION TESTS - 2 OF 4	RBL-3-1512
A26	LL 60 RANDOM STAGE II RECORD CONTROL R, R̄, AND GRADATION TESTS - 3 OF 4	RBL-3-1513
A27	LL 60 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 4 OF 4	RBL-3-1514
A28	LL 65 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 1 OF 5	RBL-3-1515
A29	LL 65 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 2 OF 5	RBL-3-1516
A30	LL 65 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 3 OF 5	RBL-3-1517
A31	LL 65 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 4 OF 5	RBL-3-1518
A32	LL 65 IMPERVIOUS STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 5 OF 5	RBL-3-1519
A33	LL 65 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 1 OF 3	RBL-3-1520
A34	LL 65 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 2 OF 3	RBL-3-1521

SUPPLEMENT A CONT'D

<u>Plate No.</u>	<u>Title</u>	<u>File No.</u>
A35	LL 65 RANDOM STAGE II RECORD CONTROL R, R, AND GRADATION TESTS - 3 OF 3	RBL-3-1522
A36	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 1 OF 8	RBL-3-1523
A37	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 2 OF 8	RBL-3-1524
A38	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 3 OF 8	RBL-3-1525
A39	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 4 OF 8	RBL-3-1526
A40	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 5 OF 8	RBL-3-1527
A41	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 6 OF 8	RBL-3-1528
A42	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 7 OF 8	RBL-3-1529
A43	LL 70 RANDOM STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 8 OF 8	RBL-3-1530
A44	BERM ZONE STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 1 OF 4	RBL-3-1531
A45	BERM ZONE STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 2 OF 4	RBL-3-1532
A46	BERM ZONE STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 3 OF 4	RBL-3-1533
A47	BERM ZONE STAGE II RECORD CONTROL R̄, R, AND GRADATION TESTS - 4 OF 4	RBL-3-1534
A48	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 1 OF 8	RBL-3-1535
A49	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 2 OF 8	RBL-3-1536
A50	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 3 OF 8	RBL-3-1537
A51	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 4 OF 8	RBL-3-1538
A52	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 5 OF 8	RBL-3-1539
A53	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 6 OF 8	RBL-3-1540
A54	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 7 OF 8	RBL-3-1541
A55	LL 55 IMPERVIOUS STAGE I RECORD CONTROL Q & S TESTS - 8 OF 8	RBL-3-1542
A56	LL 55 IMPERVIOUS STAGE II RECORD CONTROL Q & S TESTS	RBL-3-1543
A57	LL 60 RANDOM STAGE I RECORD CONTROL Q & S TESTS	RBL-3-1544
A58	LL 60 IMPERVIOUS STAGE II RECORD CONTROL Q & S TESTS	RBL-3-1545
A59	LL 60 RANDOM STAGE II RECORD CONTROL Q & S TESTS	RBL-3-1546

SUPPLEMENT A CONT'D

<u>Plate No.</u>	<u>Title</u>	<u>File No.</u>
A60	LL 65 IMPERVIOUS STAGE II RECORD CONTROL Q & S TESTS - 1 OF 2	RBL-3-1547
A61	LL 65 IMPERVIOUS STAGE II RECORD CONTROL Q & S TESTS - 2 OF 2	RBL-3-1548
A62	LL 65 RANDOM STAGE II RECORD CONTROL Q & S TESTS - 1 OF 2	RBL-3-1549
A63	LL 65 RANDOM STAGE II RECORD CONTROL Q & S TESTS - 2 OF 2	RBL-3-1550
A64	LL 70 RANDOM STAGE II RECORD CONTROL Q & S TESTS - 1 OF 2	RBL-3-1551
A65	LL 70 RANDOM STAGE II RECORD CONTROL Q & S TESTS - 2 OF 2	RBL-3-1552
A66	BERM ZONE STAGE II RECORD CONTROL Q & S TESTS	RBL-3-1553
A67	LL 55 IMPERVIOUS STAGE I RECORD CONTROL COMPACTION TESTS - 1 OF 2	RBL-3-1554
A68	LL 55 IMPERVIOUS STAGE I RECORD CONTROL COMPACTION TESTS - 2 OF 2	RBL-3-1555
A69	LL 55 IMPERVIOUS STAGE II RECORD CONTROL COMPACTION TESTS	RBL-3-1556
A70	LL 60 RANDOM STAGE I RECORD CONTROL COMPACTION TESTS	RBL-3-1557
A71	LL 60 IMPERVIOUS STAGE II RECORD CONTROL COMPACTION TESTS	RBL-3-1558
A72	LL 60 RANDOM STAGE II RECORD CONTROL COMPACTION TESTS	RBL-3-1559
A73	LL 65 IMPERVIOUS STAGE II RECORD CONTROL COMPACTION TESTS	RBL-3-1560
A74	LL 65 RANDOM STAGE II RECORD CONTROL COMPACTION TESTS	RBL-3-1561
A75	LL 70 RANDOM STAGE II RECORD CONTROL COMPACTION TESTS	RBL-3-1562
A76	BERM ZONE STAGE II RECORD CONTROL COMPACTION TESTS	RBL-3-1563

PERTINENT DATA

GENERAL

Location of Dam	One mile south of I-470 and 1/2 mile east of Raytown Road. In SW 1/4 NW 1/4, Sec. 4, T. 47 N., R. 32 W., Jackson County, Missouri, at approximately 109th Street, Kansas City, Missouri, and at mile 42.9 on the Little Blue River.
Purposes	Flood control, recreation, water quality fish and wildlife.
Authorization	Flood Control Act of 1968 (Public Law 90-483, 13 August 1968)
Operating Agency	Corps of Engineers, Project Manager offsite
Drainage Area Above Longview Dam	50.3 square miles
Approximate Length of Lake	At multipurpose level (El. 891.0 ft., m.s.l.): from the dam, about 3.5 miles in a southeasterly direction.
Approximate Length of Shoreline	24 miles in length at multipurpose level (elevation 891.0 ft. m.s.l.)
Time of Water Travel at Bank-full Rates	13 hours to the Lake City gauge. 21 hours to the Missouri River.
Maximum Flood of Record Near Damsite	42,300 c.f.s., 13 August 1982
Channel Capacity	1,000 c.f.s.
Top of Dam Elevation	926.6 feet, m.s.l.
Length	1,900 feet
Type of Construction	Rolled earthfill
Fill Quantity	1,028,000 cubic yards impervious 915,000 cubic yards random
Freeboard	3.71 feet
Maximum Height Above Valley Floor	100 feet
Top Width	74 feet
Maximum Base Width	900 feet

PERTINENT DATA (CON.)

Date of Closure	16 June 1983
Date Storage Began	16 September 1985
Date Multipurpose Pool Initially Filled	23 September 1986

SPILLWAY

Type	Uncontrolled limited service
Location	Left abutment
Crest Elevation	911.3 feet, m.s.l.
Width	200 feet
Discharge Capacity	Top, surcharge pool El. 922.9 feet NGVD 22,970 c.f.s.

OUTLET WORKS

Location	Left abutment
Type	Uncontrolled hooded drop inlet
Conduit	5.5 foot by 5 foot arch
Length	916 feet
Invert Elevation at Intake	816.0 feet, m.s.l.
Invert Elevation at Outlet	812.0 feet, m.s.l.
Discharge Capacity at Elevation 922.9 (Top Surcharge Pool)	1,040 c.f.s.
Discharge Capacity at Elevation 909.0 (Top of Flood Control Pool)	990 c.f.s.
Emergency Drawdown Gate	1-6.0 foot by 7.0 foot hydraulically operated slide gate
Low Flow Intake	2-2.0 foot Dia knife gate valve
Low Flow Outlet	2-2.0 foot Dia knife gate valve

The outlet works also contain dual 54-inch sewers with two 54- by 54-inch sewer gates and two 54- by 54-inch crossover gates.

# PERTINENT LAKE ELEVATION

<u>Storage Designation</u>	<u>Elevations (feet m.s.l.)</u>		<u>Storage Capacity* (acre-feet)</u>		<u>Area (acres)</u>
	<u>From</u>	<u>To</u>	<u>Initial</u>	<u>100-year</u>	<u>Top of Pool</u>
Surcharge	909.0	922.9	35,400		3,200
Flood Control	891.0	909.0	24,800	24,300	1,960
Multipurpose	816.0	891.0	22,100	20,600	930
Gross Storage	816.0	909.0	46,900	44,900	
Sedimentation Reserve			2,000		
Annual Sediment Inflow			20		

\*100-year sediment allocation initially allocated 1/4 (500 acre-feet) to flood control zone and 3/4 (1,500 acre-feet) to the multipurpose zone.  
Sediment allocation computed by 40 acre-feet per square mile per 100-year.

Fee Taking Line	Elevation 914.0 feet, m.s.l.
Fee Lands Acquired	4,762 acres
Easement Lands Acquired	63 acres

OPERATION AND MAINTENANCE MANUAL

LONGVIEW LAKE

LITTLE BLUE RIVER BASIN

LITTLE BLUE RIVER, MISSOURI

APPENDIX V

CHAPTER 1

INTRODUCTION

1-01. Purpose and Scope of Report. The purpose of this report is to provide in one volume the significant information needed by engineers to familiarize themselves with Longview Lake, reevaluate the embankment in the event unsatisfactory performance occurs, and provide guidance for designing comparable earth dams. The scope of this report provides a summary record of significant design data, design assumptions, specification requirements, construction equipment, construction procedures, construction experience, field control and record control test data, and embankment performance monitored by instrumentation during construction and during initial lake filling.

1-02. Project Purpose. Longview Lake is a multipurpose project realizing benefits from flood control, water quality control, recreation, and fish and wildlife.

1-03. Project Authorization. Longview Lake was authorized by the Flood Control Act of 1968 as a part of the "project for flood control and other purposes on the Little Blue River in the vicinity of Kansas City, Missouri, was authorized substantially in accordance with the recommendations of the Chief of Engineers in House Document No. 169, Ninetieth Congress First Session."

## CHAPTER 2

### PROJECT DESCRIPTION

2-01. Project Location. The Longview damsite is located on the Little Blue River at approximately 109th Street, Kansas City, Missouri. The entire lake is located in Jackson County, Missouri. Location and vicinity map are shown on Plate 1.

2-02. Dam Description. The dam is a zoned, compacted earthfill embankment with a crest length of approximately 1,900 feet and approximately 110 feet in height. The crest width is 74 feet to accommodate the 109th Street which is a 4-lane divided highway. A central impervious core with an impervious cutoff trench to bedrock was used for the total length of the dam. The embankment includes an inclined pervious drain as well as a pervious blanket across the valley and up the abutments. Compacted random zones and berm zones are used to support the impervious zones and add stability to the embankment. The theoretical top of dam is 926.6 which provides for 3.9 feet of freeboard above the maximum surcharge pool. However, the dam was constructed to elevation 925.7 with the remainder of the freeboard being provided by the New Jersey type concrete median barrier. The upstream slope is designed to have graded riprap in the zone of frequent fluctuation (elevations 878-898) and at the top 10 feet of the dam (elevations 915-925). Slope protection consists of 18-inch riprap on 9-inch of bedding. Between elevations 898 and 915 a grassed slope protects the embankment. The general embankment plan and sections are shown on Plates 2 and 22 through 25, respectively. A clay blanket was constructed on the right upstream abutment between elevations 848 and 898. A small dike was constructed west of Raytown Road near Elm Avenue to fill in a saddle in the left abutment. This dike is only about 3 to 4 feet high with a crest width of 10 feet.

2-03. Spillway. The spillway is an uncontrolled limited service spillway 200 feet wide with 1V on 6H side slopes cut in the left abutment overburden. The invert elevation is 911.3 which is 2.3 feet above the full pool elevation.

2-04. Outlet Works. The outlet works consist of an intake tower with a covered uncontrolled service drop-inlet, an arch-shaped conduit through the embankment, a parabolic drop into a stilling basin with an end sill and an outlet channel to the Little Blue River. The conduit crosses the dam centerline at station 98+70 and is skewed 3 degrees towards the downstream right abutment from the perpendicular to the dam axis. The concrete arch-shaped conduit is 4.83 foot wide and 5.5 foot high. Two 54-inch diameter sewer pipes encased in concrete are located below the conduit. The tower becomes submerged at elevations above 900 and a bridge at that elevation provides access to the tower from Service Road A.

2-05. Construction History. Since Longview Dam was located in a very urbanized area, there were many utility relocations which had to be completed prior to construction of the dam. The most extensive relocation was the existing 48-inch diameter sewer within the reservoir area which also crossed the dam centerline. It had to remain in service throughout the construction of the dam. It was unknown during the design phase exactly when the Little Blue Sewer District would complete their portion of the new sewer line within the reservoir area. Because of the uncertainty and the need to ensure closure



was not made until the sewer construction was complete within the reservoir area, construction of the dam was divided into two stages. The Stage I construction began in September 1979 and was completed October 1982. It consisted of foundation excavation for the cutoff trench and the outlet works including the outlet channel; foundation grouting; construction of the intake tower, conduit-sewer structure, and stilling basin; existing sewer relocation, diversion, and removal; placement of portions of the embankment on both sides of the river; and a 200,000 cubic yards impervious stockpile to be used for the Stage II contract. (See Plate 17 for Stage I construction.) The Stage II construction began in August 1982 and was completed September 1985. This contract included diversion of the river and embankment closure, completing the embankment, excavation the spillway, and construction of access and service roads. A 6-month construction halt was also included in the Stage II contract when the embankment reached elevation 890.

2-06. Lake Filling. After closure was made but before the embankment was topped out a lake formed several times due to excess runoff in the drainage basin. The highest lake level recorded during construction was elevation 865 on 10 June 1984 with the embankment at approximately elevation 885. This lake level was reached in less than 24 hours with the gates fully open. The embankment was topped out in September 1984 and the gates were closed to begin impoundment on 16 September 1985. The Initial Reservoir Filling Plan DM No. 20 was used as a guide during filling. The desired filling rate was 5 feet per month after the pool reached elevation 850. In October 1985 and November 1985 the pool rose faster than was acceptable and the emergency drawdown gate was opened to draw the lake down to the desired filling curve. On 23 September 1986 the lake reached multipurpose pool. The projected and actual filling curves are shown on Plate 45.

## CHAPTER 3

### GEOLOGY

3-01. Regional Geology and Physiography. Longview Lake is located in the extreme northern part of the Osage Plains Section of the Central Lowlands Physiographic Province. The Osage Plains Section is characterized by old scarp plains beveling faintly inclined strata. The Dissected Till Province which borders immediately to the north of the project is characterized by submaturely to maturely dissected till plains. A maturely dissected drainage pattern developed during pestilences time trends north into Missouri River. The project is located on the southern boundary of the Kansan Age Glacial Stage. Scattered loess deposits up to 7 feet thick are present on the uplands in the area. These deposits thicken to up to 90 feet along the Missouri River bluffs.

3-02. Site Geology. Gently rolling uplands surround Longview Lake. The Little Blue River flows generally north in a meandering, incised channel cut through alternating limestone and shale strata. At the damsite the valley is 1,800 feet wide from bluff top to bluff top and topographic relief is about 120 feet. The Bethany Falls limestone member of the Kansas City Group, Missourian Series, Pennsylvanian System is a prominent scarp forming ledge in the reservoir area. Its thickness is about 18 feet and it dips gently from elevation 885 on the right abutment to elevation 876 on the left abutment. Bedrock on the lower valley walls and valley floor is made up of less resistant shale and siltstone of the Pleasanton Group.

3-03. Description of Overburden. Overburden on the uplands in the general area consists of up to 30 feet of residual clay and scattered deposits of wind blown loess up to 20 feet thick. No loess deposits were encountered at the damsite. Overburden on the valley side slopes consists of residual clays and colluvium with clayey gravel at the base of the slopes. Alluvium in the valley ranges from 12 to 29 feet with the greatest thickness occurring in the terrace on the left side of the valley. The alluvium consists of predominately lean clay with lesser amounts of fat clay and silty, sandy gravel, in the basal 7 feet, resting on shale and siltstone bedrock. Generally the clays have medium to stiff consistencies although some soft clays were encountered in a few borings.

3-04. Bedrock. Bedrock at the damsite consists of alternating beds of limestone and shale of the Kansas City Group overlying a more massive section of less resistant clayey and silty shale, siltstone and sandstone beds of the Pleasanton Group. These rocks are classified as lower Pennsylvania Series. Underlying the Pennsylvanian rocks are about 2,200 feet of older sedimentary strata. Only the lower one-half of the Kansas City Group and the upper one-third of the Pleasanton group are exposed at the damsite. See the General Geologic Column and Legend on Plate 6. The regional dip of the bedrock ranges from 10 to 20 feet per mile west, northwest, but local variations in dip and strike occur. At the damsite bedrock dips 1 foot in 300 feet to the east into the right abutment and 1 foot in 65 feet to the west into the left abutment.

Jointing of beds may be very intense locally, due to dome-related flexures. Primary joints are generally vertical and strike north 50 degrees east spaced 5 to 35 feet apart. Secondary joints are generally, vertical,

tight, strike North 75 degrees West and spaced about 2 feet apart. A series of faults are known to exist in the road cut along a one mile reach of Highway I-470, located about 3/4 mile north of the damsite. However, faulting could not be traced into the soil profile and no evidence of faulting was found in the cutoff trench or foundation excavations during construction.

Depth and degree of weathering vary considerably with rock type and thickness of soil cover. The limestones are softened and stained to depths of a few inches to 5 feet below their surface. The Bethany Falls crops out prominently along the valley wall and partially filled joints, up to 5 feet in width, are apparent at the outcrop. By 50 feet into the abutment the effects of weathering of the Bethany Falls appears to have been moderated by the presence of overlying shale. Large blocks of Bethany Falls measuring up to 15 feet or so in their greatest dimension became detached at the joints due to the weathering and erosion processes with some blocks moving down slope. Neither the Winterset nor the Sniabar limestone formed prominent outcrops in the abutments and were commonly mantled by a 1 foot to 8 foot depth of overburden. Weathering effects extended as much as 12 feet beneath the bedrock surface of the interbedded limestones and shale. A more detailed description of the bedrock and bedrock units can be found in the Construction Foundation Report.

3-05. Subsurface Investigations Prior to Construction. Geological investigation of the damsite included field reconnaissance, review of geologic literature, analysis of air photos, survey of water wells, review of gas and oil well logs and exploratory drilling. A total of 339 borings and 11 test pits were completed by government drill crews prior to Stage I construction. Overburden exploration included 2-inch, 4-inch, and 6-inch diameter drive holes; 3-inch and 5-inch Shelby tube samples, auger samples, and samples obtained from test pits. The bedrock investigation included NQ, NX, 4-inch and 6-inch cores. Eleven NX, angled, core holes were drilled on the left abutment and five were drilled on the right abutment. Thirty-three bedrock borings were hydraulically pressure tested. Natural gas was encountered in hole DC-10 and C-145A between elevations 762 and 768. Gas pressure measured 11 psi with the packer set at elevation 801. A 5-minute pressure test with 35 psi yielded a "take" of 0.2 gallons per minute. The gas in both holes was found to be at low pressures and limited in volume as it quickly dissipated during drilling. A .5-foot cavity was logged in siltstone 13 feet below the top of the bedrock surface in hole UC-9. Two additional holes were drilled in the vicinity of the void; however, neither the cavity nor the evidence as to the geologic cause could be found.

## CHAPTER 4

### FOUNDATION AND EMBANKMENT

4-01. General. The main embankment consists of a compacted earthfill approximately 1,900 feet long with an outlet works and an uncontrolled limited service spillway (200 feet base width) located in the left abutment. The maximum height of the embankment above the valley floor is 110 feet. Theoretical top of dam is 926.6, but the embankment was only constructed to elevation 925.7 and the remaining freeboard is achieved by the New Jersey type median barrier. The dam was constructed with a centrally located impervious core to prevent seepage through the embankment. The impervious core is supported on either side with random and berm zones. The various embankment zones were designed to utilize the available borrow material. An inclined pervious drain was constructed on the downstream side of the impervious core and is connected to a horizontal blanket which extends along the base of the embankment and exits at the downstream toe. Any seepage which may occur through the impervious core is intercepted by the inclined drain and carried out the embankment through the horizontal blanket. The foundation overburden was relatively shallow with a layer of clayey or silty sandy gravel located directly above the bedrock. In order to cutoff any water flowing through the basal gravels an impervious cutoff trench was constructed into rock for the full length of the dam. Various foundation bedrock formations were highly jointed and fractured. A grout curtain was installed below the cutoff trench for the full length of the dam in order to control the seepage through the bedrock. A plan view is shown on Plate 2.

a. Valley Section. (Station 90+00 to 97+00) The embankment across the main valley varies from 100 to 120 feet in height with an average height of 110 feet above the valley floor. Zoning consists of centrally located impervious core supported by random and berm zones. Typical valley section is shown on Plate 23. The foundation overburden consisted of between 12 and 29 feet of alluvium which was predominantly lean clay with lesser amounts of fat clay. A clayey or silty sandy gravel was common in the basal portion (1 to 7 feet) of the alluvium. The underlying bedrock surface was Pleasanton Zone E (argillaceous siltstone).

b. Right Abutment. (Station 86+25 to 90+00) The right abutment embankment section is similar to the valley section except that the downstream slope is flattened to 1V on 6H, there is no downstream berm zone, and an impervious clay blanket was placed upstream on the far right portion of the abutment. Typical right abutment sections are shown on Plates 22 and 23. The natural slope of the upper portion of the abutment between approximate stations 86+00 and 87+50 was about 1V on 8H. From approximate station 87+50 to the toe of the abutment at station 90+00 the slope was irregular and considerably steeper, averaging 1V on 2.8H. The foundation overburden on the slopes consisted of residual clays and colluvium, while at the base of the abutment clayey gravel underlaid alluvial clay. Thickness of the overburden ranged from zero at the Bethany Falls outcrop to a maximum of about 15 feet at the base of the valley wall. The foundation bedrock consists of 12 stratigraphic units of interbedded limestone and shales (Plate 7).

c. Conduit Section. (Station 98+00 to 100+00) The conduit section is located basically at the toe of the left abutment (Plate 24). The centerline

of the conduit crosses the dam axis at station 98+70 and is skewed 3 degrees towards the downstream right abutment from perpendicular to the dam axis. The conduit section is constructed of impervious material with no random or berm zones. Since an all impervious zone with higher strength clays was used, the embankment slopes were steepened allowing for a shorter sewer-conduit structure. The overburden thickness on the right side of the conduit varied from 16 to 27 feet and consisted of alluvium similar to that in the valley section. At the toe of the left abutment from approximate station 99+00 to 100+00, the overburden was colluvium and clay alluvium consisting of clay, and fragments of shale and limestone ranging in size from gravel and cobbles to slabs and large blocks. The overburden rested on Pleasanton Zone C (argillaceous limestone) from the upstream end of the approach channel to near the dam axis. Except for a short horizontal extension of the Pleasanton Zone D (calcareous siltstone) the top of bedrock from the dam centerline to the downstream end of the outlet channel was Pleasanton Zone E (argillaceous siltstone). The outlet works profile is shown on Plate 8.

d. Left Abutment. (Station 100+00 to 107+00) This section is similar to the valley section except there is no upstream or downstream berm zone and the downstream slope is flattened to 1V on 6H. A typical left abutment section is shown on Plate 24. The natural slopes of the left abutment were generally not as steep as those of the right abutment. Between approximate stations 100+00 and 100+65, the average slope was 1V on 4H and it flattened to 1V on 6.5H between stations 100+70 and 103+00. The natural slope became progressively flatter between station 103+00 and the end of the embankment. From approximate station 100+00 to station 100+65 overburden ranged in thickness from 6 to 22 feet and consisted essentially of colluvium similar to that in the conduit section and residual clays. From the top of the Bethany Falls outcrop at approximate station 100+70 to the end of the embankment at station 107+00, the overburden consisted of residual soils of essentially lean and fat clays. Thickness of overburden ranged from 2 to 16 feet with the greater depths underlying the flatter upland slope. The foundation bedrock consists of the same 12 stratigraphic units of interbedded shales and limestones found on the right abutment (Plate 7).

#### 4-02. Foundation Treatment.

a. Valley section. All of the embankment foundation was stripped of boulders, sod, roots, tree stumps, etc. and the overburden disked to a minimum depth of 8 inches and compacted with a minimum of three passes of a rubber tired roller. The 48-inch diameter reinforced concrete sewer pipe and pipe bedding, which crossed the dam axis at station 97+95, was removed under the entire embankment. Cleanup of the sewer excavation was accomplished and the backfill consisted of impervious under the impervious zone and random elsewhere.

b. Outlet Works Foundation. All overburden below the outlet works was excavated and backhoes and airspades were used to excavate the rock. A rock saw was used at the tower to excavate the vertical sides of the tower foundation. The final foundation in the outlet works except for the intake tower and stilling basin was excavated with a "roto mill" which consisted of a large track-mounted cutting roller. Air water jets were used to clean the final bedrock surface prior to placing a 6-inch layer of lean concrete. Lean concrete was also used downstream of the centerline of the dam in order to

meet the required grade for the outlet works. The lean concrete was as much as 4-feet thick at the lower end of the outlet works and stilling basin.

c. Abutment Foundations. The subcrop faces of several limestone ledges were trimmed back to either a 1V on 1H or 1V on .5H slope not only at the centerline but for a distance of as much as 400 upstream and 235 downstream. In some areas concrete bulkheads were placed against the bedrock faces to seal open bedding planes and vertical joints to achieve the desired slope to place fill (Photo 35). In other areas the open joints on top of the horizontal benches were gravity grouted (Photo 19). A total of 279.75 sacks of cement were used to dry pack joints or as poured grout at 35 separate locations. A total of 342.1 cubic yards of lean concrete was placed behind forms to seal joints and maintain the required slopes.

d. Cutoff Trench. In order to control seepage through the foundation overburden and to facilitate foundation grouting, the cutoff trench was excavated to bedrock from station 86+00 on the right abutment to station 104+00 on the left abutment. Side slopes in the overburden are 1V on 2H and in the bedrock are excavated to 1V on 1H. The floor of the cutoff trench is 30 feet which allows 24 feet of impervious and a 6 foot pervious drain backfill. Prior to curtain grouting, the cutoff trench was excavated to grade with the exception of the exposed shales which were left 2 feet high in order to protect the final shale surface from weathering or drying out. Grouting was performed from the floor of the cutoff trench through 2-3/4 inch steel pipes cemented 2 feet into the bedrock. Once the curtain grouting was complete the steel grout pipes were removed. The upper 2 feet of grout holes in the limestone were gravity grouted while the upper 2 feet of shale was removed to bring the shale surface to final grade. The final bedrock surface was cleaned with air-water jets. The cutoff trench backfill was performed in the dry utilizing small gasoline pumps in sumps. The main sources of seepage were small areas encountered within the basal clayey gravels. Details of the dewatering efforts are included in Chapter 8.

4-03. Embankment Materials. The embankment zoning was designed to economically use the available material from required excavation and upstream borrow, to provide seepage control, and to provide slope stability. Because of the distribution of materials in the borrow area and the need to maximize the strength of fill placed in the conduit section and in the lower portions of the embankment considerable effort was expended in defining quantities of various liquid limit materials in the borrow. For the most part the easily available lower liquid limit materials were in the low lying areas and were used in the Stage I embankment or were stockpiled to avoid inundation after closure. The remainder of the borrow had CL and CH materials interlayered and had to be differentiated to insure optimum use of higher strength material. See also Chapter 8.

a. Impervious. The specifications for Stage I and II required the impervious material consist of soil classified as CL and CH with the material placed upstream of the dam axis having no more than 5 percent gravel. For Stage I, the maximum liquid limit specified for all impervious material was 55. The maximum liquid limit specified for the Stage II impervious material varied from 55 to 65 depending on the embankment feature and elevation. Generally, the lower liquid limit material was placed in the lower portions of the embankment. The Stage II impervious zone liquid limits are shown on the table below:

## STAGE II IMPERVIOUS ZONE LIQUID LIMITS

<u>Area</u>	<u>Maximum Liquid Limit</u>
Below elevation 867, including cutoff trench	55
Between stations 97+00 and 101+00 - Below elevation 887	55
Cutoff Trench above elevation 867	60
Inspection Trench	60
Between stations 97+00 and 101+00 - Above elevation 887	60
Impervious Clay Blanket	65
Above elevation 867, excluding cutoff trench, inspection trench, and between stations 97+00 and 101+00	65

Typical cross sections showing the impervious zone liquid limits are on Plate 46. The allowable moisture content at the time of placement ranged from 2 percent below optimum to 3 percent above optimum. It was required that the impervious material be compacted to at least 95 percent maximum density obtained by the standard Proctor compaction test. Most of the impervious material was placed in 8-inch lifts and compacted with 6 passes of a sheepsfoot roller except the material directly over the foundation rock. (See Chapter 8 for procedure used.) A total of 1,028,528 cubic yards of impervious were placed in the dam with 509,866 cubic yards being placed in Stage I and 518,662 being placed in Stage II.

b. Random. The random material consisted of overburden material except that soil classified as OH, PT, MH, and OL. The random material came from the required excavations and upstream borrow. Some of the random zone consisted of gravelly material not suitable for the impervious zone. Maximum liquid limit in the Stage I contract was 60, but varied between 60 and 70 in the Stage II contract. Lower maximum liquid limits were required in the lower portions of the embankment. The Stage II liquid limits are listed below:

### RANDOM ZONE LIQUID LIMITS

<u>Area</u>	<u>Maximum Liquid Limits</u>
Below elevation 867 upstream of dam axis	60
Below elevation 867 downstream of dam axis	65
Above elevation 867	70
Spillway Dike and Elm Avenue Dike	*

\* No liquid limit requirement but shall be a CL or CH material.

Plate 46 shows the various liquid limit zones on typical cross sections. Moisture requirements for random were the same as for impervious and compaction was either six passes with a sheepsfoot roller on 8-inch lifts or three passes with a rubber-tired roller on 12-inch lifts. A total of 914,813 cubic yards of random material was placed in the dam with 215,552 cubic yards being placed in the Stage I contract and 699,261 cubic yards being placed in the Stage II contract.

c. Berm. The upstream and downstream berm consisted primarily of fat clay (CH), however, according to the specifications could have been shale and other materials from the required excavations and borrow areas unsuitable for or in excess of the requirements for impervious, random, or rockfill. Fat clay (CH) was used in the upper 5 feet of berm normal to the upstream slope and for the closure Phase I and II cofferdams. The only moisture content requirements for Stage I was to the extent required for unassisted movement of compaction equipment. For Stage II the requirements were the same as Stage I on the wet side of optimum but on the dry side of optimum were limited to no more than 4 percent below optimum. On Stage I, the berm was compacted using two passes of a rubber-tired roller with 24-inch lifts or two passes of a sheepsfoot roller with an 8-inch lift. For the Stage II contract compaction was performed with either four passes of a sheepsfoot roller on an 8-inch lift or two passes of a rubber-tired roller on a 12-inch lift. A total of 965,971 cubic yards of berm were placed with 344,467 cubic yards being placed in Stage I and 621,504 cubic yards being placed in the Stage II contract.

d. Pervious. The pervious material was clean free-draining, durable, natural sand obtained from a commercial Missouri River sand plant having the following gradation:

<u>Sieve Size</u>	<u>Percent by Weight Passing</u>
No. 4	90-100
No. 16	55-85
No. 50	5-20
No. 200	0-5

The majority of the sand had less than .5 percent passing the 200 sieve. Stage I required the pervious material be compacted using 3 passes of a vibratory roller on 12-inch lifts. For hand compacted (special backfill) a plate vibratory roller on 4-inch lifts to obtain 80 percent relative density was required. During construction this was reduced to 70 percent (see paragraph 8.02 d.(2)). The compaction procedures for Stage II were the same as Stage I and based on the previous experience the required relative density was reduced to 70 percent for a plate vibratory compactor with 6-inch lifts. Moisture content of the sand was controlled such that the sand was saturated during compaction. A total of 104,584 cubic yards of pervious was placed with 30,387 cubic yards being placed in the Stage I contract and 74,197 cubic yards being placed in the Stage II contract. The outlet of the pervious zone is protected with a 12-inch thick layer of bedding which was placed using a track-mounted backhoe. The bedding has the following gradation:

12-Inch Bedding

<u>Dimension Sieve Size</u>	<u>Percent by Weight Passing</u>
6-inch	maximum allowable
3-inch	70-90
1-1/2-inch	55-75
1/2-inch	35-55
#4	15-35
#10	0-20
#20	0-5



e. Rockfill. The better quality limestone excavated from the abutments was used as rockfill (photo 21) to add stability and protection to the upstream clay blanket. The specification required the rockfill be reasonably well graded with a maximum size stone of 300 pounds and no more than 10 percent passing the 2-inch sieve. Shale and other soft friable particles were limited to 10 percent. The rockfill was placed in 36-inch lifts with no moisture control or compaction requirements except when the rockfill was placed within 2 feet of bedrock, berm or impervious clay blanket, at which time it was to be wheel-rolled with approved equipment.

f. Special Backfill. Special backfill is placed in the bedrock irregularities or other confined areas adjacent to structures where rollers could not operate effectively. The 2-foot thick zone of material against the bedrock surfaces was also considered as special backfill. This included the impervious and pervious zones for Stage I and the impervious and random zones for Stage II. The specifications required the special backfill be placed in 3-inch lifts. The impervious and random material was compacted with a power tamper to 95 percent of standard Proctor density, and the pervious was compacted with a plate vibrator to 80 percent relative density which was later reduced to 70 percent. The moisture requirements for Stage I were the same as the corresponding zone of material and for Stage II were between optimum and 3 percent above optimum. The rate of placement was controlled so that the relatively thin hand-compacted zones were covered with equipment-compacted material as soon as possible to prevent drying, excess wetting, or freezing and thawing.

g. Channel Fill. Material from required excavation and stockpiles which were not suitable for the embankment were used as channel fill. Rock excavation including shale was also allowed as channel fill as long the rock was covered with at least 2 feet of soil material. This material was placed in the old river channel downstream of the embankment with no moisture control except that required to allow traffic compaction of the 24-inch maximum lifts.

4-04. Stone Protection. The upstream stone protection consists of 18-inch riprap overlaying 9-inch bedding located between elevations 878 to 898 and 915 to 925. The bedding and riprap gradations are as follows:

#### 9-Inch Bedding

Dimension Sieve Size	Percent by Weight Passing
5-inch	maximum allowable
3-inch	75-95
1-inch	40-60
1/2-inch	20-40
#4	0-20

### 18-Inch Riprap

Weight in Pounds Per Stone	Percent of Total Weight Lighter Than
400	100
200	60-90
100	30-50
25	0-15

All of the riprap and bedding was placed during the Stage II contract using either a track-mounted backhoe or a track-mounted front-end loader. (This equipment was not allowed on the placed stone.) (See Photos 54 and 55.) The upstream riprap is Bethany Falls limestone which was obtained from the Orrick Stone Co. Quarry in Clay County, Missouri. The upper 4.5 feet of material in the quarry was not used because of the poor quality rock.

The upstream riprap was designed in accordance with "Criteria for Riprap Wave Protection in Missouri River Division" dated June 1974 and ETL 1110-2-221 (29 Nov 76). The riprap design and the basis for the riprap limits are shown on Plate 32. The riprap was sized based on the design wave height (HD) which is equal to the significant wave height (HS) multiplied by 1.4. Significant wave height (HS) for the lower slope protection was 2.2 feet generated from 59 m.p.h. wind with a 20 minute duration. The critical combination for the upper area of stone protection is 56 m.p.h. wind for 30 minutes causing a significant wave height (HS) of 2.9 feet. The theoretical riprap necessary for the upper riprap zone is 21-inch, however, 18-inch was used instead since there is a very remote possibility the design condition will ever occur.

Based on economics, the embankment slope was designed to minimize the amount of required slope protection. The upstream riprap was placed between elevations 878 and 898 and elevations 915 and 925. These stone protection zones were determined from the Pool Elevation-Duration Curve, Lake Stage Frequency Curve and the chart showing the number of day per hundred years the pool is within the wave-height of a given elevation. In addition the upper slope has riprap to reduce wave run-up and protect the upper embankment during a full flood pool event.

The stilling basin riprap was also 18-inch and was placed similarly to the upstream slope. The stilling basin riprap is Winterset limestone from the lower 8.5 feet of the Courtney Road Quarry in Jackson County, Missouri. The riprap was designed based on methods described in ETL 1110-2-8 and ETL 1110-2-221. The design parameters used were an average flow velocity of 5.6 ft./sec. over the end sill, a design wave height of 2.8 feet, a safety factor of 1.5, and a specific gravity of 2.6. The riprap was designed based on wave height requirements, since the waves would cause more damage than the velocities.

4-05. Settlement. Settlement analyses were performed on the foundation overburden based on consolidation test results of select samples. These studies indicated a maximum total settlement of 2.2 feet in the valley foundation overburden and 1.9 feet in the terrace overburden. It was anticipated the majority of this settlement would occur during construction with only minor amounts occurring after construction. A 0.6 foot overbuild was constructed to compensate for post-construction settlement. During Stage

II construction, three settlement plates were installed in the valley section to measure settlement during and after construction. Two of the settlement plates are installed in the foundation overburden and one is installed in the embankment just above the pervious blanket in the random zone. As of April 1990, the maximum settlement in the two foundation plates is .71 foot with .66 foot occurring during construction and the settlement in the lower embankment plate is .8 foot with .54 foot occurring during construction. Also seven crest settlements monuments were installed to monitor settlement after construction. The crest settlement monuments indicate uniform settlement across the valley no more than .35 foot.

## CHAPTER 5

### SEEPAGE CONTROL

5-01. Through Seepage. Seepage through the embankment is controlled by a combination of embankment zoning and an inclined pervious drain. The impervious core prevents excessive seepage through the embankment. The random zones also assist in preventing through seepage since it was constructed primarily of impervious material. Any seepage which does occur is intercepted by the inclined pervious drain located on the downstream edge of the impervious core. The inclined pervious drain is connected to the horizontal sand blanket and together they will draw the phreatic surface down well within the embankment.

5-02. Underseepage. Seepage beneath the embankment is controlled by a cutoff trench, grout curtain and the extension of the sand blanket up the abutment. The impervious cutoff trench extends through the overburden and weathered rock down to firm bedrock. The grout curtain was constructed from the bottom of the cutoff trench and extends the full length of the dam.

a. Cutoff Trench. The cutoff trench was constructed to control seepage through the foundation overburden and weathered rock. It extends the full length of the embankment from station 86+00 on the right abutment to 104+00 on the left abutment. The trench is 30 feet wide along the bottom with excavated side slopes of 1V on 2H in the overburden and 1V on 1H in the bedrock. The cutoff trench was backfilled with impervious material to prevent seepage under the embankment. However, a 6 foot wide pervious drain which is connected to the horizontal sand blanket was placed on the downstream side of the trench to intercept any seepage which may occur.

b. Grout Curtain. The grout curtain was constructed to prevent underseepage through the bedrock. It consists of three lines of grout holes in the abutments and a single line of grout holes in the valley. Grouting was performed from the floor of the cutoff trench through 2 3/4-inch diameter steel pipe cemented about 2 feet into bedrock. The grout holes were drilled the total depth using rotary drilling methods. After drilling, the grout holes were washed through the drill rods, pressure tested, and grouted through a packer which was set at successively shallower depths. The grout pressure was not allowed to exceed the estimated vertical stress at the packer depth. All the grout holes were inclined 30 degrees from vertical and drilled parallel to the dam centerline into the abutments. The grouting plan and profile are shown on Plates 14, 15 and 16. Some variations to this plan were made during construction. A more detailed description of the actual grouting plan and operations are provided in the Construction Foundation Report. Line A is 10 feet downstream of the dam axis and actually extends from station 86+25 to 89+65 on the right abutment and between station 100+05 and 103+85 on the left abutment. Line B is located 10 feet upstream of the dam axis and was constructed from station 86+20 to 89+65 on the right abutment and 100+05 to 103+85 on the left abutment. Line C is along the dam axis and extends from station 84+00 to 112+00.

c. Abutment Seepage. In addition to the cutoff trench the grout curtain, and the sand blanket which extend up the abutments, the embankment slopes at the abutments were flared and a clay blanket was constructed on the

right abutment to control underseepage at the abutments. Excess rock from required excavation was used in an outer rockfill zone to protect the clay blanket. The majority of the abutment seepage was expected to occur through the Bethany Falls limestone rather than the relatively tight lower bedrock units. Therefore, additional readily available rockfill was placed along the abutment up to elevation 848. The clay blanket was then placed on top of the rockfill and carried up to elevation 898.

## CHAPTER 6

### SLOPE STABILITY

6-01. General. This chapter discusses slope stability investigations. It presents the design analysis and consists of slope stability which was performed in conjunction with establishing embankment slopes and internal zoning. The results of the design analysis were originally presented in Design Memorandum No. 5, Soil Data and Embankment Design dated June 1978. A stability analysis of the left embankment which was performed after Stage I construction when high construction induced pore water pressures were measured in the embankment is discussed in Chapter 8 and Enclosure 1. A stability analysis was also conducted after Stage II construction for the conduit section when movements were detected in a lower zone of the embankment.

6-02. Design Strength Testing. Laboratory tests to determine strength parameters for design were performed on samples of materials from the required excavations, proposed borrow areas, and from the foundation. The testing consisted of laboratory tests on undisturbed samples of foundation material and on remolded samples from required excavation and potential borrow material. The tests generally followed procedures outlined in EM 1110-2-1906 entitled "Laboratory Soils Testing", 30 November 1970. Compaction of remolded or disturbed strength test specimens of cohesive soils was in accordance with EM 1110-2-1902, "Engineering and Design Stability of Earth and Rockfill Dams", 1 April 1970. Test loads generally covered the range anticipated in the embankment or foundation.

a. Triaxial Compression Tests. Triaxial "Q" and "R" tests were performed on representative undisturbed and compacted samples under confining chamber pressures ranging from 0.5 to 6.0 tsf. Pore pressures were measured in some of the "R" tests for the purpose of determining the effective strength of the soil. The general procedure involved strain controlled tests on two or three specimens trimmed from a single sample horizon. Composite specimens were compacted in eight layers by kneading to the range of densities and moisture contents expected during construction. Specimen dimensions were 1.4 inch by 3.0 inch height. No attempt was made to saturate the "Q" test specimens. "R" tests were saturated with back pressure ranging from 1.9 to 6.0 tsf. The strain rates were 0.5 percent per minute for the "Q" tests and 0.12 percent per minute for the "R" tests. In "R" tests, consolidation under the assigned chamber pressure was continued until a plot of volume change versus log time indicated that primary consolidation was complete. Peripheral filter paper strips were used to accelerate drainage. The deviator stress at 15 percent strain was assumed to be the maximum if it had not peaked beforehand.

b. Direct Shear Tests. Direct shear tests were performed on representative undisturbed and compacted soil specimens and on undisturbed shale specimens. Composites were compacted by kneading to the assigned density and moisture content, generally 95 percent of standard effort maximum dry density at a moisture content 3 percent greater than optimum. Normally, two specimens were sheared under a normal load of 4, 5, or 6 tsf. Before shear, the specimens were allowed to consolidate under the applied normal load until primary consolidation was complete. Specimens were sheared at a constant rate of 0.0003 inches per minute. Normally, this rate of deformation

met the 50t<sub>50</sub> criterion. Residual shear tests on intact and pre-cut samples were performed with a direct shear machine capable of shearing the specimen, then reversing the direction of shear 180 degrees and shearing the specimen again without releasing the normal load in the process. In order to obtain the strength at large strain, this procedure was repeated until no reduction in shear strength occurred from additional shearing.

6-03. Selection of Design Strengths. In selecting design values, considerations were given to 1) stress-strain compatibility, 2) the relationship of maximum shear strength to ultimate shear strength, 3) the number of tests, 4) the nature of the material, and 5) the location of the material with respect to potential failure surfaces. Design shear strengths were selected with two-thirds of the test values exceeding the design values.

a. Foundation Overburden. Test results of the terrace and valley reaches showed that one design strength was appropriate for the entire valley section. The following shear strengths were selected.

<u>TYPE OF STRENGTH</u>	<u>c (TSF)</u>	<u>Tan <math>\phi</math></u>
Q	0.60	0.00
R	0.20	0.20
S	0.00	0.50

b. Foundation Shales. Only "S" strengths were used for shales in the design. Unconfined compression tests indicate that the "Q" strength is substantially greater than the overburden and therefore is not a factor in the stability analysis. Past experience in the Kansas City District indicates that because these shales are highly overconsolidated, "R" strength is generally greater than "S" strength over the range of normal stresses encountered. The design "S" strength for the shales in the abutments and valley is  $c = 0$ ,  $\tan \phi = 0.60$ . A residual "c" strength of  $c = 0$ ,  $\tan \phi = 0.30$  was also used.

c. Compacted Embankment Material. In the original DM analysis higher strength lean and fat clays with liquid limit less than 55 from the low-lying borrow were to be used in the impervious zone and in the conduit section between stations 97+00 and 101+00. The higher liquid limit fat clays (with LL generally less than 70) from the upper borrow areas were expected to be used in both the impervious and the random zones for the remainder of the embankment. Thus the same strengths were assigned to the impervious and random zones for these reaches. (As a result of comments from higher authority, this was changed during the plans and specifications so that the lower liquid limit materials were used in the impervious zone to the extent possible and liquid limit restrictions were also applied to the random zone. The objective was to use the lowest available liquid limit materials in the lower part of the embankment to optimize strength and stability of the embankment. For a distribution of material requirements developed during the plans and specification phase of the design see Chapter 4.) The design strengths used in the DM stability analysis are shown in the following table. The pervious zone had a higher strength than the random or impervious but because of the limited extent of this zone the higher strength was ignored in the stability analysis.

#### RANDOM AND IMPERVIOUS

<u>TYPE OF STRENGTH</u>	<u>c (TSF)</u>	<u>Tan <math>\phi</math></u>
Q	0.90	0.00
R	0.20	0.20
S*	0.00	0.50
S**	0.00	0.36

\* Liquid limit less than 55, conduit section.

\*\* Higher liquid limit material from higher elevation borrow areas.

#### BERM ZONE

<u>TYPE OF STRENGTH</u>	<u>c (TSF)</u>	<u>Tan <math>\phi</math></u>
Q	0.45	0.00
R	0.10	0.20
S	0.00	0.30

6-04. Design Stability Analysis. The embankment was analyzed for stability at three locations in an effort to cover the various conditions existing in the dam. Those locations are the valley section, conduit section, and the cofferdam.

a. Method of Analysis. A force equilibrium wedge method was used to analyze the slope stability of the Longview embankment. A computer program titled "Stability Analysis - Wedge Method" (File No. 741-F5-cl020), commonly known as KC Wedge, was used to perform the stability analysis. This program satisfies force equilibrium only using an analytical technique discussed on Plate 86. It searches for the shear surface with the minimum factor of safety. The results were checked by searching from more than one starting point to make sure that the program did not miss potential failure areas. The slope of the effective "E" force was assumed to be one-half the average outer embankment slope. The computer program is compatible with Appendix VI of EM 1110-2-1902, except for: (1) the slope of the "E" force is assumed to be constant throughout the active wedge, and (2) the angle of the shear surface was assumed to be the same for each material in the active wedge. The critical shear surfaces were checked manually. In all cases, the manual checks confirmed the computer factors of safety. Typical analyses for the end of construction, partial pool, rapid drawdown from maximum surcharge and full pool, and steady seepage cases are discussed below and are shown on Plates 87 and 88.

b. Construction Case. The design analyses assumed that neither the embankment nor the foundation consolidates during construction. Submerged weights were used below and moist weights above the line of saturation, which was assumed near the ground surface. Upstream and downstream slopes were analyzed using "Q" (unconsolidated-undrained) shear strengths and were designed for a minimum factor of safety of 1.3. In order to meet the required 1.3 factor of safety in the valley section a construction halt at elevation 915 was needed to allow consolidation of the foundation clay. This was later



changed to elevation 890 and embankment fill placement rate restrictions were established (see Chapter 8).

c. Partial Pool Case. The upstream slopes were analyzed for pool elevations ranging from near the valley floor to near full pool to determine the critical pool level. The intermediate shear envelope  $(S, (R+S)/2)$  was used. The saturation line was assumed to be horizontal at the pool elevation being considered. Submerged weights were used below and moist weights above the saturation line. The slopes were designed for a minimum factor of safety of 1.5 for a slide in the impervious and random zone. In the valley section a partial pool slide in the berm had a 1.22 factor of safety with the pool at elevation 855. The upstream berm slope was not flattened because of the improbability of the pool being at this elevation long enough for full saturation of the embankment to occur.

d. Steady Seepage Case. The downstream slopes were analyzed for a spillway crest pool, steady seepage case. Vertical equipotential lines were assumed. The shear envelopes and design factors of safety were the same as for the partial pool case. The slopes were also analyzed using residual strengths for the foundation shales, with a minimum factor of safety of 1.0 being required.

e. Sudden Drawdown Case. The upstream slopes were analyzed for a sudden drawdown condition, both from the spillway crest and from maximum surcharge. In both studies, it was assumed that the lake was drawn down to multipurpose pool. The combined "S" and "R" strength shear envelope was used. The analyses assumed that the embankment was saturated as shown and that the pool was instantaneously lowered with no dissipation of pore pressure occurring during drawdown. Slopes were designed for a minimum factor of safety of 1.2 for a drawdown from the spillway crest and a minimum factor of safety of 1.0 for a drawdown from the maximum pool.

f. Earthquake Case. The slopes of the embankment were subjected to a pseudo-static earthquake analysis during design. Longview Dam is located near the border between seismic zones 1 and 2. ER 1110-2-1806 requires the evaluation of potential liquefaction for all existing projects in zone 2 where embankment or foundation materials are present that are suspect of being susceptible to liquefaction or excessive deformation. Since the clay soils encountered are not vulnerable to liquefaction, a pseudo-static analysis was conducted.

g. Results. The results of the stability analyses performed for the design memorandum on the valley section, conduit section, and the cofferdam are shown on Plates 89 through 91.

6-05. Stage II Stability Analysis. Because of high construction induced pore pressures in the conduit embankment and a zone which developed deformation during Stage II construction, additional stability studies were made for the end of construction conditions. Analysis was performed on the upstream slope of the conduit section where both inclinometer data and observed bridge movements showed movement was occurring. The zone of movement roughly corresponded with the area of highest pore pressure. While pore pressures were experienced at the closure section and the right embankment, no significant movement was detected at other locations and the pore pressures

were some what less and tended to be confined to the cutoff trench backfill (See Plate 44). Although there was some indication that placement water contents were farther above optimum than indicated by the field test data (see Chapter 8), it is believed the left embankment experienced unusually high pore pressure response because of its differential foundation conditions and its relatively small size as constructed in Stage I. The pore pressure was concentrated in that portion of the embankment which was placed on relatively rigid rock foundations or conduit. The left embankment foundation has a considerable portion placed on rock or concrete: the conduit excavation, the old sewer trench excavation and the cutoff trench. It appears that the backfill, being some what stiffer than adjacent overburden, probably carries load from surrounding fill areas which would increase stresses and thus pore pressure. Both total and effective stress analyses were performed. The effective stress analysis used both a pore water pressure coefficient, Bishop's  $r_u$ , and pore pressure contours developed from measured induced construction pore pressures. Inclined meters showed a zone of movement occurring at approximately elevation 827 at range approximately 1+80 upstream and analyses were performed with the shear surface through this zone.

a. Method of Analysis. The computer program SSTAB2 (Spencer's method) was used for the Stage II analysis. It satisfies all requirements of static equilibrium by varying the safety factor and side force inclination simultaneously. This program was written by Stephen Wright, University of Texas, Austin.

b. Revised Soil Parameters. The shear strength for the soil was determined from field compaction results, record control testing, and from testing on undisturbed samples taken both during the installation of instrumentation and from the side walls of two large diameter auger holes installed at the end of Stage I construction. The unit weight used was 125 pcf. The "S" strength for the effective stress analyses was determined from record control direct shear and triaxial "K" tests with pore pressure measurements. An envelope of  $c = 0.1$  tsf and  $\tan \phi = 0.5$  was selected. "S" strengths were believed appropriate because of the movement experienced and that shear stress induced pore pressures were being measured. For the total stress analysis it was assumed a soft zone existed in the zone of movement. No attempt was made to specifically sample and test this zone since the embankment had been investigated previously during Stage I and no defined soft zone was found. Some strength test results showed strengths lower than the design value but a continuous layer was not identified. A reduced strength of 0.5 tsf was assumed for comparison purposes.

c. Results. Initial stability analysis was made using a simplifying assumption for the pore pressure of  $r_u = .65$ . This analysis showed  $F = 1.10$  which was verified with a hand solution. It was believed that the pore pressure was not this high especially in the outer and upper 25 feet of the embankment. An analysis was conducted using pressure contours developed from the piezometer readings. A circular search showed a minimum  $F = 1.63$  with the surface passing through a portion of the zone of movement. A wedge shaped shear surface through the zone showed  $F = 1.87$ . A total stress analysis with the shear surface passing through the zone of movement shows  $F = 1.24$  using the conservative strength assumption ( $c = .5$  tsf) in the zone of movement. Results of these analyses are shown on Plate 92.

## CHAPTER 7

### INSTRUMENTATION

7-01. General. The initial installation plan for observation devices included instrumentation to monitor pore pressure in the embankment and the foundation clays and shales, the effectiveness of the cutoff trench and grout curtain, the foundation and embankment settlement, and the horizontal and vertical movement of the embankment. The piezometers were to be installed in three lines at stations 93+00 (valley section), 97+00 (closure section), and 98+70 (conduit section). Based on previously experienced high construction pore pressures in the foundations at other dams in the Kansas City District, the majority of the instrumentation devices were open tube and gas operated piezometers installed in the foundation overburden and shales. Piezometers were also installed in the foundation overburden upstream and downstream of the cutoff trench to check its effectiveness and in the embankment to monitor construction pore pressure. Alignment monuments were installed to detect slope movement as the embankment was being topped out while crest settlement monuments were installed soon after the embankment was topped out to monitor vertical movement. The initial instrumentation plan included 14 open tube piezometers, 12 gas operated pore pressure cells, 7 crest settlement monuments, 3 foundation settlement plates, and 21 alignment monuments total in one upstream and one downstream line. However, because of high construction induced pore pressures in the left embankment which developed during Stage I construction, additional observation devices were installed to monitor movements and pore water pressures particularly in the embankment. The monitoring program was expanded and further complicated by the development of gas in the open tube devices and concerns for the reliability of the devices. A discussion is included in Chapter 8 and Enclosure 1. It should be noted that all devices are thought to be influenced by gas and that the magnitude of the measuring error has not been determined although considerable effort has been directed to that end. At the time of this report, the instrumentation includes: 52 open tube piezometers of various design, 22 gas operated pressure cells, one vibrating wire piezometer, 23 alignment monuments total in 3 lines, 3 foundation settlement plates, 7 crest settlement monuments, and 4 inclinometers. The locations of these devices are shown in plan view and cross section on Plates 35 through 41.

#### 7-02. Piezometers.

a. Open Tube Piezometers. A total of 62 open tube piezometers have been installed in the embankment, foundation, and abutments of Longview Dam since construction began. Ten of these instruments were abandoned; 8 because they were to be submerged upon reservoir filling, one because it was on the embankment centerline and could not be read after embankment completion, and one because it was broken 29-feet below the top of the riser and could not be repaired. There are a total of 52 open tube piezometers remaining on the project of three basic types. There are forty-one 3/4-inch PVC piezometers with slotted tips, four 3/4-inch PVC piezometers with 1/16-inch perforations, and seven 2-inch PVC piezometers with slotted tips. The open tube piezometer installation details are shown on Plate 42.

The 3/4-inch piezometers with slotted tips consist of 6.5-foot long sections of 3/4-inch inside diameter, schedule 80, PVC pipe. On early

installations, the sections of pipe were connected with stainless steel threaded couplings whereas the later installations utilized flush joint PVC pipe. All of the 3/4-inch piezometers terminate in sensing tips consisting either of a 1.5-inch diameter by 18-inch long, #20 (0.020-inch) slotted PVC pipe or a 2-inch diameter by 24-inch long, #10 (0.010-inch) slotted PVC pipe. Sand surrounds each tip and fills the borehole so as to be in contact with the zone or zones to be monitored. Where a single zone was to be monitored, a 4-foot seal of bentonite balls was placed above the sand with a 5 percent bentonite-cement grout filling the remainder of the hole. Piezometers installed to monitor several strata are all located on the right abutment and have a one-foot seal of bentonite balls above the sand with the remainder of the hole (3-foot minimum) filled with a 1 to 1 sand and granular bentonite backfill.

The 3/4-inch piezometers with 1/16-inch perforations are similar to those described above with the exception that, on 6-foot centers, 1-foot long sections of the schedule 80 PVC riser pipe were patterned with 1/16-inch holes. This pattern of perforations and the sand backfill around the riser was continued to within approximately 14 feet of the ground surface where a 1-foot thick bentonite ball seal was placed. The remainder of the hole was then filled with a 1 to 1 granular bentonite and sand backfill. All of the 3/4-inch piezometers with perforated risers are located on the abutments.

The 2-inch diameter piezometers with slotted tips are also similar to the 3/4-inch piezometers with slotted tips with the exception that, regardless of slot size, all of the tips are 2 inches in diameter. All of the 2-inch piezometers except PPE92-2 are located in the foundation overburden downstream of the right abutment and carry the prefix "W-". PPE92-2, the only 2-inch open tube piezometer in the embankment is made of glued joint PVC pipe.

Where 3/4-inch piezometers were brought up through the embankment, 5-foot lengths of 2-inch I.D. PVC protective pipe surround the 3/4-inch pipe above the original installation borehole (Plate 43). The fill was compacted directly against the protective pipe. The 5 foot sections are joined using double bell slip couplings with rubber sealing rings to accommodate movement and settlement.

Several open tube piezometers produce large quantities of gas which makes evaluation of data difficult due to widely and rapidly fluctuating water levels. The presence of gas in open tube piezometers was identified early in construction and an investigation to identify the type of gas was undertaken. The collection and identification processes are documented in Enclosure 1. The piezometers that have experienced problems with g-s are PPE92-1, PPE95-1, PPE97-13, PPE97-14, PPE99-10, and PPE99-13, all of which are located either in the upstream or downstream random or impervious zones. A 2-inch diameter open tube piezometer (PPE92-2 data plots shown on Plates 109 and 110) was installed adjacent to a 3/4-inch open tube piezometer (PPE92-1 data plots shown on Plates 107 and 108) which had been giving inconsistent data due to gas. Both devices were installed with the tips at the same elevation. The 3/4-inch device had a water level at the top of the riser and would periodically overflow and would erupt after a pressure gauge was removed. The 2-inch piezometer stabilized at a level about 20-feet below the ground surface and showed a steady decrease in water level as would be expected. In attempts to prevent the 3/4-inch devices from overflowing and to stabilize the fluctuating

water levels, pressure gauges and air release valves were installed on the risers. Although 3/4-inch risers should de-air themselves, the air release valves have not proven entirely effective and in several cases the pressures indicated by the piezometers increase substantially when a gauge and air release valve are added due to gas partially displacing water in the riser (Plates 107 and 108). The existence of gas in the piezometers and its effect on water levels is not as extreme as it has been in the past, although it does continue to be a problem.

b. Gas Operated Piezometers. A total of 32 gas operated (pneumatic) piezometers have been installed in the embankment and foundation of Longview Dam, of which, 22 remain functional. Three instruments were destroyed by a 1982 flood, two were destroyed by construction equipment, four were abandoned because of water in the pressure leads or a diaphragm failure, and one indicated pressures above the range of the readout device. Of the 22 remaining gas operated piezometers at Longview, 18 are Sinco Model 51471 pore pressure transducers (2-tube) with "Norton" (low air entry) filters, 3 (PPE99-8, PPE99-12, and PPE99-15) are Petur model P-100 pneumatic piezometers (low air entry filter), and 1 (PPE99-11) is a Sinco Model 51471 pore pressure transducer with a "Coors" (high air entry) filter. PPE99-8 and PPE99-11 were installed within 5-feet horizontally and 12-feet vertically of each other for comparison purposes (Plates 116 through 119).

Each of the three types of gas operated piezometers were installed differently and the installation details are shown on Plate 42. The standard Sinco 51471 transducers were installed in 3.5 feet of saturated sand in a 6-inch diameter borehole. Above the sand, the hole was sealed with 4.5 feet of bentonite balls. The remainder of the hole was then filled with a 1 to 1 mixture of sand and granular bentonite. The Petur P-100 pressure cells were installed in existing 3/4-inch I.D. PVC open tube piezometers. The transducer was placed in the PVC riser and 8 feet of saturated sand was placed in the riser. A 1-foot thick plug was then formed around the pressure leads inside the riser using a water sensitive expanding polymer. The riser was cut off 1 foot below the surface of the fill under construction and the remainder of the riser was filled with sand. A 1-foot thick bentonite ball seal was placed over the top of the cut off PVC riser and embankment construction continued over the bentonite seal. The Sinco 51471 with the high air entry Coors filter was installed in an existing 2-inch diameter PVC piezometer. The 2-inch riser was filled with saturated sand to within 8 feet of the surface of the embankment. Bentonite balls were then placed in the remaining 8 feet of riser to form a seal. Again, embankment construction continued above the bentonite seal.

Five of the existing gas operated piezometers have begun, or continue, to provide erratic readings and have water in the pressure tubing at least intermittently. These problems are believed to be due to the piezometers and the associated tubing approaching the end of their useful life expectancy. The possibility of continuing to obtain useful readings from these devices is questionable. Difficulties have been encountered in reading the following devices: PPE93-1, PPE93-6A, PPF97-9, PPF99-1, and PPE99-6.

c. Vibrating Wire Pressure Cell. The vibrating wire piezometer, PPE99-18, is a Geokon Model 4500. The vibrating wire piezometer cell was placed in a 1.5-inch diameter vertically slotted PVC pipe filled with

11 pounds per gallon kaolinite slurry (Plate 42). The leads to the cell were extended from the top of the 1.5-inch diameter pipe through a length of 3/4-inch PVC pipe. This length of pipe was also filled with kaolinite slurry and sealed 4.7-feet above the piezometer tip with a plug of water sensitive expanding polymer. A 6-inch diameter leather collar was placed around the 3/4-inch pipe at the same location as the polymer seal. This entire assembly was then lowered to the bottom of a 6-inch borehole filled partially with the kaolinite slurry using a 0.4-foot long length of 3/4-inch PVC pipe to keep the unit off of the bottom of the borehole. A 4-foot seal of bentonite balls was placed above the leather plug and a 1.5-foot layer of saturated sand placed above that. The remainder of the borehole around the 3/4-inch pipe containing the piezometer leads was filled to the ground surface with a 5 percent bentonite-cement grout. The readout leads for this device are entirely encased in 3/4-inch PVC pipe between the top of the 1.5-inch diameter slotted pipe and monitor box 5 on the downstream crest road shoulder.

7-03. Embankment Piezometric Devices. Following completion of the Stage I left embankment, high pore pressures were measured in the lower part of the embankment. Between the Stage I and Stage II contracts, overflowing water levels and quick recoveries after pumping were found in the protective casings of several devices in the closure section. It appeared that the protective casings and the risers were acting as separate measuring devices. The protective casings were apparently registering higher pore pressures at points higher in the embankment, possibly near the mid-height. These pressure levels apparently exceeded 50 percent response. An extensive investigation was undertaken to determine the extent of the high pore pressures and the adequacy, including stability, of the embankment. The investigation involved undisturbed sampling and testing, inspection of in place embankment material from two large diameter auger holes, installation of additional instrumentation to monitor pore pressures and slope movement, dye injection, and pump tests. A complete discussion of the investigation is contained in Enclosure 1. The investigation revealed construction induced pore pressures in the left embankment backfilled excavations below approximate elevation 810 ranging up to about 75 percent. Between approximate elevations 820 and 830, above the backfilled excavations, pore pressure responses were between 50 and 60 percent. No pore pressures were present in the top 20 feet of the Stage I fill.

Most of the piezometers installed in the embankment measured induced construction pore pressures after completion of the embankment and continue to do so. In general, piezometers located in the cutoff trench and adjacent to the conduit registered the highest pore pressure levels upon embankment completion with some indicating pore pressure of up to 63 percent, or about 65-feet above the top of the dam.

Construction pressures in the embankment remain high especially in the cutoff trench and around the conduit (40 to 55 percent pore pressure response). Most of the piezometers are showing a pressure dissipation of about 1 to 5-feet per year in the embankment although some have not begun to show pore pressure dissipation.

Piezometers installed in the embankment indicate pore pressure levels above the pool elevation, therefore the effect of pool filling on the embankment piezometers has yet to be determined.

All piezometers on the project are currently being read quarterly.

7-04. Foundation Piezometric Devices. During construction, little pore pressure developed in the foundation shale or overburden. During reservoir filling, piezometers located in the foundation overburden on the upstream side of the cutoff trench indicated nearly full reservoir head which was apparently being transmitted through the basal gravel layers. The piezometers located in the upstream foundation overburden still indicate nearly full reservoir head, however, no significant pressures are being transmitted downstream. The only other foundation piezometer to show high pressures was PPF99-4 with its tip in the Pleasanton Zone E formation (Plates 114 and 115). It built up pore pressure to an elevation of 880 shortly after construction was completed and has dropped to elevation 837 as of the fall of 1990. Since filling of the reservoir in 1986, all of the piezometers in the Pleasanton Zone E siltstone on the upstream side of the grout curtain have shown a slow, but constant, increase in piezometric level. These increases have been anywhere from 8 to 14 feet with PPF86-1A on the upstream right abutment showing the largest increase (Plate 102). The piezometers in the Pleasanton Zone E siltstone on the downstream side of the grout curtain have shown general decreases in measured pore pressure from 1 to 14 feet except for PPF87-3 as noted in the "Abutment Piezometric Devices" section below.

7-05. Abutment Piezometric Devices. Piezometers located in the right abutment bedrock, downstream of the grout curtain, rose as the pool filled lagging the pool generally on the order of 14 feet. This prompted the installation of piezometers in various geologic members to determine which ones were transmitting pressure. Some pressure testing and bail tests were done as the holes were being drilled. It was found the formations transmitting the high pressures included the Pleasanton Zones A & B, as well as the Middle Creek limestone and Hushpuckney shale. Piezometers were also installed in the pervious drain after excessive water was noted exiting the pervious blanket. Piezometer readings indicated the pervious blanket was completely saturated and even pressurized in some areas. It was concluded that the excessive water was coming from seepage through the right abutment bedrock. No other problems have surfaced due to these high pressures and it appears that seepage through the right abutment is being controlled and collected by the pervious blanket.

The piezometers in the Pleasanton Zone E siltstone on the downstream side of the grout curtain have shown general decreases in measured pore pressure from 1 to 14 feet except for PPF87-3 (Plate 103). PPF87-3 has shown a 3 foot increase in the measured piezometric level in the Pleasanton siltstone since 1986. This increase is most likely due to migration of high pressures from the overlying Hushpuckney shale, Middle Creek limestone, and the Pleasanton Zones A and B.

Seepage pressures remain high in the right abutment bedrock and are influenced by the pool. A steady flow continues to exit the pervious blanket as a result of this seepage. This causes the toe ditch to remain wet and causes a maintenance problem in this area. No other problems have surfaced due to these high pressures.

7-06. Inclinometers. There are three inclinometers installed through the upstream slope and one through the downstream slope. All are between stations 97+00 and 99+00 (Plate 36). All of the inclinometers are constructed of 3.38-inch outside diameter, grooved aluminum casing (Plate 43A). The casing was installed in 10-foot lengths and joined with 3.63-inch outside diameter by 12-inch long telescoping aluminum couplings. The casings were pop-riveted to the couplings with aluminum rivets and a gap was left between subsequent casings to allow for settlement of the foundation or embankment. In the foundation overburden, a 12-inch gap was left between casings while a 6-inch space was left between casings that passed through the embankment. Each inclinometer is sealed at the bottom with a pop-riveted plastic cap and protected at the surface of the fill by a 6-inch diameter, lockable, galvanized steel pipe.

I-97-1 and I-99-2 were installed in 7-inch flight augured boreholes through the partially completed embankment and foundation overburden. The bases of the inclinometers are seated in holes drilled using a 5-5/8 inch rock bit in the foundation rock. Pea gravel was used as backfill material in the annulus around the inclinometer casings. A short length of 6-inch diameter steel pipe filled with pea gravel was placed around the inclinometer at the surface of the fill. As the embankment was brought up, the fill was compacted directly against the 6-inch pipe. After each few lifts, the pipe was raised to the new fill surface leaving the pea gravel in direct contact with the compacted embankment material. This process was repeated until the final surface of the fill was reached where a 1 to 1 cement grout seal was placed. I-99-1 was installed after completion of the embankment at the device, in a borehole drilled entirely with a 6-1/8 inch rockbit through the embankment, foundation, and into the foundation rock. Pea gravel was placed in the annulus around the inclinometer casing and a 4-foot long seal of bentonite balls was placed around each coupling. I-99-3 was also installed after completion of the embankment adjacent to the bridge abutment and the borehole was made using a 5-7/8 inch rockbit exclusively. A 1:1 cement grout fills the annulus between the inclinometer casing and the borehole walls.

Inclinometer I-99-1 was installed at station 95+40, range 2+57 downstream (Plate 41) in September of 1984, one month prior to topping out of the embankment. This device has shown a total movement of 4.7 inches most of which occurred between elevations 803 and 829 prior to September of 1986 (Plate 121 and 122).

Inclinometers I-97-1 (station 97+49, range 1+89 upstream) and I-99-2 (station 98+25, 0+85 upstream) were installed after the Stage I embankment was completed and were extended through the embankment as it was constructed (Plate 41). I-97-1 has shown a total upstream movement of 7.5 inches most of which occurred between elevations 823 and 842 (Plate 124).

I-99-2 has shown approximately 8.2 inches of upstream displacement, most of which took place in the top 4 feet of the fill and between elevations 819 at the bottom of the inclinometer and 856 (Plate 122). Most of the movement at the base of the device occurred as the embankment was being topped out or shortly after completion of the embankment. The movement shown in the top 4 feet of the embankment is attributable to the placement of a concrete work pad on the up slope (downstream) side of the device. The concrete pad was in contact with the protective pipe around the inclinometer and has caused the



top section of the inclinometer to lean upstream. Neglecting the movement shown in the top 4 feet of the embankment, I-99-2 shows 6 inches of total upstream movement.

Since October of 1987, I-99-2 has been experiencing a problem with "silting in". Between October of 1987 and September of 1990, the sounded depth of this inclinometer decreased 15 feet from 108.8 to 93.8 feet. This "silting in" is reflected by the progressive upward shifting of the bottom of the readings on the movement versus depth plots shown on Plate 122. The loss of the lower 15 feet of the instrument is also evidenced by the shifting of the data plots to the right as the incremental movement shown in the lower portion of the instrument is lost. The exact nature of the material in the inclinometer and the cause for the silting is currently unknown. Flushing of the casing and sampling of the material is planned. If it is found that substantial quantities of embankment materials are found in the instrument, it will be abandoned by grouting the casing full.

I-99-3 was installed at station 98+50, range 1+80 upstream adjacent to the tower bridge abutment (Plate 41). During and after construction, this device indicated lateral spreading of the embankment in the conduit section on the order of 4.5 inches (Plates 123 and 124). This spreading resulted in movement of the control tower bridge abutment and complete closure of the expansion joints at both the bridge abutment and pier. Most of the movement occurred between elevations 816 and 837 with a fairly concentrated zone of 1.9 inches of incremental movement at elevation 826.3. As a result of this movement the embankment stability was checked (see Chapter 6). Repair of the bridge abutment and reestablishment of the tower bridge expansion joints was accomplished by contract in the spring of 1990.

No substantial movement has been shown by any of the inclinometers since 1987. All four inclinometers are currently being read twice a year.

7-07. Alignment Monuments. The alignment monuments on the downstream line "A" and upstream line "B" consist of 18-inch diameter by 8-foot deep reinforced concrete monuments (Plate 43). Alignment line "A" is located 200-feet downstream of centerline between stations 87+50 and 101+00 and contains 8 monuments on 200-foot centers. Line "B" is located 170-feet upstream of the embankment centerline between stations 87+00 and 103+50 and contains 9 monuments on 200-foot centers (Plate 36). In addition to alignment monuments, each line has two instrument monuments; one on each abutment.

As previously noted, lateral spreading of the embankment in the conduit section caused movement of the control tower bridge abutment and complete closure of the expansion joints at both the bridge abutment and pier. Alignment line "C" was installed to monitor the tower bridge abutment movement and consists of only one alignment plate and two instrument monuments (Plate 43). The alignment plate is located at station 98+66, range 1+77 upstream, and is installed on the north end of the east bridge parapet over the abutment. There is not a concrete monument installed at that location, only a stainless steel instrument plate mounted on the bridge parapet. Line "C" uses the same left abutment instrument monument as the upstream line "B". A separate instrument monument for line "C" exists on the right abutment upstream of the monument for line "B" to allow a straight shot across the bridge abutment.

Initial readings on all three lines were taken three times and the results averaged to establish a zero reading. The initial readings of lines "A" and "B" were taken in July of 1985. The initial reading of line "C" was taken in March of 1986 after installation of the alignment plate. "A" on the downstream slope has experienced the most overall movement with a maximum total movement of 1.7 centimeters at monuments A-5 and A-6 at stations 95+00 and 97+00 respectively (Plate 98). The maximum total movement of 1.4 centimeters of the upstream line is shown by monument B-3 at station 91+00 (Plate 98). Vertical surveys taken on each monument indicate maximum settlements since installation on the downstream line at monument A-6 which shows a total settlement of 0.19 feet (Plate 99). The maximum settlement on the downstream line is 0.14 feet at monument B-4 at station 93+00 (Plate 100). Line "C" has indicated no movement or settlement since installation since all bridge abutment movement took place prior to installation of the monuments. All alignment monuments on the project are currently being surveyed semiannually.

7-08. Foundation Settlement Plates. Three mechanical foundation settlement plates were installed during construction in 1983. These devices were installed at stations 91+00, 93+00, and 96+00 and are downstream of the embankment centerline 40, 50, and 75 feet respectively (Plate 36). SPPF91-1 and SPPF93-7 consist of a 36-inch square, 1/2-inch thick steel plate set approximately 1-foot below the embankment/foundation contact (Plate 43). The plate of SP96-1 is also a 36-inch square steel plate but it is installed approximately 10-feet above the pervious blanket as opposed to below the blanket at the foundation contact as is the case with the other settlement plate installations. The location of the plate allows SP96-1 to monitor settlement in approximately the lower 13 feet of the embankment. To allow monitoring of changes in elevation of the plates, a 1-inch diameter standard galvanized pipe is connected to a flange bolted to the plate. The 10 inches of galvanized pipe immediately above the plate is perforated with 12, 1/16-inch holes to allow monitoring of pore pressures as well as settlement. (The use of all three settlement plates to monitor pore pressures was discontinued in May of 1986 due to the presence of heavy oil in the riser.)

As noted on the plots, there was no reliable settlement data recorded between August of 1984 and May of 1985 since the contractor failed to record a riser extension (Plate 97). It was assumed that no settlement took place during this period. The three settlement plates indicate that the bulk of foundation consolidation had occurred by October of 1984 with only small settlements since that time. The range of settlements indicated is between 0.5 and 0.8 feet with SPPF96-1 showing the greatest settlement. The maximum current rate of settlement shown by the settlement plates is less than 0.1-foot per year. See section "Crest Settlement Monuments" below for discussion and comparisons of the foundation settlement plates and crest settlement monuments. All foundation settlement plates on the project are currently being surveyed annually.

7-09. Crest Settlement Monuments. On completion of the dam in 1985, seven crest settlement monuments (CSM's) were installed in a single line on 200-foot centers along the crest of the dam (Plate 36). The monument line runs along the inside edge of the downstream guardrail of 109th street at approximate range 0+36 downstream. The monuments are 6-inch diameter by 6-foot deep auger holes filled with reinforced concrete. The monuments are

capped with a standard brass monument cap set flush with the bituminous roadway surface (Plate 43). Four of the brass monument caps were damaged by snowplows in 1987 and it may become necessary to relocate the crest settlement monuments to the downstream side of the guardrail to prevent this type of damage from recurring.

As noted above, the crest settlement monuments were installed in July of 1985 and have since shown between 0.1 and 0.35 feet of settlement (Plates 95 and 96). CSM-4 at station 94+50 over the old river channel shows the maximum crest settlement. The crest settlement monuments are currently being read once a year and currently indicate settlement rates of less than 0.1-foot per year.

CSM's 2, 3, and 5 are located at roughly the same respective stations as settlement plates SP91-1, SP93-7, and SP96-1. Comparisons of the total settlements shown by CSM's 2 and 3 and settlement plates SP91-1 and SP93-7 indicate that approximately 70 percent of the total settlement that has occurred since 1985 has been embankment settlement. As noted above, the plate for SP96-1 is installed approximately 10-feet above the pervious blanket which allows it to monitor settlement in approximately the lower 13 feet of the embankment. Settlements indicated by CSM-5 and SP-96-1 are closer in agreement than the other comparisons and it is indicated that approximately 90 percent of the embankment settlement shown by the crest settlement monuments is occurring in the lower portions of the embankment which are not monitored by SP91-1 and SP93-7.

7-10. Top of Dam Profiles. Surveys are made annually of the top of the New Jersey barriers and the top of the dam at the downstream edge of the barriers between the lanes of 109th street across the dam (Plate 94). As noted previously, the roadway surface across the dam is below the theoretical top of dam elevation of 926.6 by approximately 1.2 feet and the New Jersey barriers are considered to account for the remaining required freeboard. The elevations taken after the September 1989 repair of the low spot on the left abutment are shown in the top plot on Plate 94.

The maximum settlement shown by the top of dam surveys occurs at station 94+50 over the old river channel and is approximately 0.3 feet. This data confirms the location and magnitude of settlement shown by the crest settlement monuments.

## CHAPTER 8

### CONSTRUCTION NOTES

8-01. General. Major sewer construction work in the reservoir area was a significant concern in planning for the construction of the dam. Closure of the embankment could not occur until the sewer was complete because of the potential to inundate the sewer construction area within the reservoir. Since it was unknown at the time of preparation of plans and specification when the sewer construction would be complete, the District decided to construct the dam in two stages. This created some problems with trying to minimize the number of temporary slopes and "tie-ins," however, a construction plan was developed with this objective in mind. The plan was complicated not only by the existing sewer which crossed the proposed foundation and the necessity for the sewer to remain in service at all times, but also by the relatively narrow valley and the fact that the best available upstream borrow was located in low-lying areas next to the river channel. To minimize "tie-ins" the plan proposed building the minimum embankment necessary in Stage I prior to diversion and closure in Stage II. This embankment was to be only that necessary for the cofferdam (except the closure section). Stage I required construction of the outlet works (conduit and sewer), diversion of the sewer, and removal of the existing sewer. It required excavations for the outlet works including the outlet channel and a river diversion channel. Cutoff trench excavation and foundation grouting was included in Stage I as well as backfilling the cutoff trench for the right and left embankments. To minimize the amount of low-lying impervious material that had to be stockpiled, it was decided to include placement of the downstream conduit embankment to elevation 867. Stage II construction included diversion of the river, construction of the closure cofferdam, the remaining portions of the downstream embankment, and raising the entire embankment to final elevation. The construction plan allowed a considerable portion of the Stage II embankment to be constructed without temporary slopes or "tie-ins." However, the configuration of the Stage I embankment and the cofferdam was not ideal since it consisted of many small pieces.

8-02. Stage I Contract. The Stage I contract, DACW41-79-C-0114, was awarded to W. A. Ellis Construction Company, Kansas City, Missouri, on August 24, 1979, for the low bid of \$11,865,103.00. Construction began in September 1979 and was completed in October 1982. Ellis Construction Company's previous experience was primarily in highway and sewer construction projects, they had never constructed a dam, nor did anyone with the company have any dam building experience. This lack of experience was compounded, especially early in the contract, by the fact that the Government did not have a resident engineer on the job full-time. The project was administered by an area engineer who visited the project only periodically and an inadequate staff. This led to immediate problems with rock excavation, foundation treatment and preparation, care of rock surfaces and especially material usage. Early in the project the specifications were often totally ignored. After these administrative problems became apparent a full-time resident engineer was assigned to the project and was given a very capable staff. The project ran much better in that the specifications were being enforced; however, the early problems led to hostile relations between the contractor and the resident office. Strict interpretation and enforcement of the specifications led to a substantial claim involving a variety of issues, the most significant of which related to

material usage from the borrow. Additionally, because of these problems and unexplained observations of the left embankment and instrument monitoring devices, serious concerns for the integrity of the Stage I embankments surfaced. The embankment experienced relatively high pore pressures apparently related to fill placement which tended to be wet of optimum and the rather complex configuration of the embankment pieces. The high pore pressures were complicated by an unexplainable accumulation of gas in the monitoring devices. An extensive investigation was conducted to determine the stability and adequacy of the Stage I embankments. The borrow area problems, embankment investigation and other pertinent features are discussed below.

a. Borrow. Much of the low liquid limit material needed for the impervious fill and the stockpile for Stage II was easily available only in, close-in, low-lying areas along the river. The other areas of the borrow contained some low liquid limit material however, it was usually overlain by high liquid limit material unacceptable for use as impervious. On previous dam construction projects, as on this one, the Government had done enough exploratory work to ensure that sufficient quantities of the desired material were available in the borrow; but usually this exploratory work was not sufficiently detailed for the actual construction. Detailed explorations were expected of the contractor; specifications clearly state this as the contractor's responsibility. However, the contractor did no exploratory work to classify material in the borrow and placed a large quantity of the close-in low liquid limit materials in the berm and random fill zones early in the job. He then began experiencing problems finding acceptable material for use in the remaining impervious zone in the left embankment and to complete the impervious stockpile required for Stage II construction. This material could not be readily classified visually because there was not a discernible difference in color and the material typically ranged in liquid limit between about 45 and 60. Visual classification techniques using toughness of the PL thread are not precise enough for most individuals to differentiate between materials with LL just below 55 from those just above 55. The contractor was directed to do exploratory work in the borrow; however, not enough quantity was found to complete either the left embankment or the impervious stockpile. An analysis of the borrow area, and of materials placed in the berm and random zones for the claim revealed that more than enough impervious had been available in the borrow to meet the fill quantities required. In fact, the analysis showed that some 180,000 cubic yards of impervious was placed in the berm zone, and 75,000 cubic yards in the random zone. Only 40,000 cubic yards had to be deleted from the Stage I contract quantities for the left embankment and the impervious stockpile.

b. Outlet Works Excavation.

(1) Conduit-Sewer. The conduit-sewer excavation began in the fall of 1979. The floor of the excavation was 25-feet wide upstream of the cutoff trench and 30-feet wide downstream of the cutoff trench (see Plate 12 for excavation plan). The overburden excavation averaged 20-feet deep with 1V on 2.5H slide slopes. The majority of the excavated overburden was used for the impervious stockpile. The rock excavation averaged only 2 to 4-feet deep and was performed with backhoes, jackhammers, or dozers with the exception of the intake tower area where a rock saw was used to make the necessary vertical cuts. The rock surface was brought to final grade using a "roto mill," a large track-mounted cutting roller. Air-water jets were used to clean off the

final bedrock surface prior to placing the specified 6 inches of fill concrete. It was required the bedrock be excavated to firm rock, so in some places additional fill concrete had to be placed to bring the final fill concrete surface to grade. The contractor elected to over excavate the east slope to provide for a 30-foot wide workbench along the tower area (O.W. station 45+00 to 46+00) and 20-foot wide bench along the conduit section. During the conduit-sewer excavation several slides occurred along the left abutment.

(a) Sewer Excavation Slide. On November 18, 1979 a slide occurred on the left side of the sewer excavation between outlet works (O.W.) stations 55+00+ to 57+00+. The slide moved on a slickensided surface in the weathered shale which underlies the talus. The horizontal movement at the head of the scarp was in excess of 8 feet and the depth to the failure plane varied from a few feet to 8 or 10 feet. The failure took place just after the excavation had been made at the toe of the natural abutment slope. (See Photo 13.) After the initial failure, additional exploratory holes were drilled and the slide area was analyzed and monitored. Through the winter of 1980, additional movement was experienced and the slide area increased. Because of the continued movements, there were concerns regarding long-term stability of the area. It was felt the slide would not adversely effect the stilling basin or the outlet works; however, continued movement would not be tolerable to the relatively rigid new sanitary sewer pipe to be located on the left side of the outlet works. Therefore, the proposed new sanitary sewer was relocated to the right side of the outlet works by Modification 6. The new sewer alignment is shown on Plate 12. The slide area was repaired by removing the slide material from the excavation and backfilling the sewer excavation to elevation 820 with the excavated overburden from the left abutment as part of Modification 16. The backfill was placed in 8-inch lifts, compacted with even routing of hauling equipment and the final surface sloped to drain away from the abutment.

(b) Left Abutment Slides. During the winter of 1980, three additional shallow slides occurred in overburden on the abutment side of the conduit excavation between O.W. stations 50+00 and 52+00. These slides were each about 50-feet wide and extended about halfway up the slope from the base of the excavation. Then again in December 1980 another slide occurred in the same area. In order to prevent further sliding of the left abutment overburden, the contract was modified to require all left abutment overburden between the cutoff trench and the downstream toe of the embankment be excavated to rock.

(2) Outlet Channel. In the late summer of 1980, the outlet channel alignment was modified to avoid the guy wires of a newly relocated power line. The outlet channel was moved toward the east downstream of O.W. station 60+41. This new alignment also eliminated the need to relocate an existing gas line.

(3) Approach Channel. The approach channel parallels the new sewer relocation which was constructed under separate contract concurrently with Stage I construction. Originally the entire approach channel excavation was to be performed during Stage II to prevent the confusion of having separate contracts in the same area. However, the majority of the approach

channel excavation was performed by the sewer contractor during the installation of the adjacent sewer. Therefore, only minor excavation was necessary during Stage II to bring the approach channel to grade.

c. Sewer Relocation. Prior to constructing the embankment, the existing 48-inch sanitary sewer located on the right and parallel to the outlet works had to be diverted through the relocated sewer beneath the conduit without interrupting service and then removed. Construction of the dual 54-inch sewer pipes located under the conduit and the removal of the existing sewer pipe within the limits of the dam were included in the Stage I contract. The remaining sewer relocation work in the reservoir area and downstream from the dam was performed under a separate sewer contract. As previously discussed, the new sewer alignment originally located on the left side of the stilling basin was moved to the right side by Modification 6. Because of this alignment change the new and existing sewers crossed. Therefore, prior to constructing the new sewer, the existing sewer had to be temporarily taken under the new sewer by an inverted siphon which was added to the contract by Modification 37. The conduit-sewer structure was completed by the fall of 1980 but the actual sewer diversion did not take place until July 1981. The existing sewer and all of the bedding material was removed shortly after diversion. The majority of the existing sewer was located on bedrock so the excavation and foundation cleanup performed was similar to the cutoff trench. The contractor excavated and cleaned up to 50 feet at a time and used air jetting for final cleanup of the shale. The sewer excavation was to be backfilled with impervious (LL<55), however, the contractor was also allowed to use fat clay. The downstream pervious blanket was placed directly over the sewer backfill rather than along the excavation limits. All of the existing sewer pipe was removed except the inverted siphon. It was left in-place and pumped full of sand to prevent the water flowing through the sand blanket adjacent to the conduit from building up pressure upstream of the new sewer.

d. Cutoff Trench.

(1) Excavation. The cutoff trench was excavated to bedrock between station 86+00 and 104+00. The floor of the excavated trench is 30-feet wide and the side slopes are 1V on 2H in the overburden and 1V on 1H in the bedrock. (See Plate 11). The contractor began excavating the trench along the valley, then proceeded to the left abutment and finished the trench excavation with the right abutment. The overburden thickness varies from 0 feet on the abutments to as much as 29 feet in the valley. The majority of the excavated overburden was placed in the impervious stockpile or temporarily stockpiled for future use in Stage I. The rock excavation was performed primarily by backhoes or presplit blasting to minimize damage to the rock surface. The specifications restricted blasting within 100 feet of concrete or grout less than 7 days old or within 50 feet regardless of age. Therefore, the majority of the blasting along the cutoff trench was completed prior to curtain grouting and all of the blasting on the left abutment cutoff trench was completed prior to placing the outlet works structural concrete. As previously mentioned the cutoff trench was excavated to grade prior to grouting with the exception of the shales which were left 2-feet high to protect the underlying shale from drying out and weathering. All curtain grouting was performed from the floor of the cutoff trench (Photo 14) and is discussed in more detail in the Construction Foundation Report. The cutoff trench excavation and curtain grouting was performed in the dry. The river

had to be temporarily diverted to perform cutoff trench excavation and curtain grouting within the river channel. The contractor made little or no effort to control surface runoff or groundwater in the cutoff trench between grouting operations and backfilling.

(2) Backfilling. The cutoff trench was backfilled with impervious fill and a 6-foot wide pervious drain on the downstream side of the trench (see Photo 16). In mid June 1980, the contractor began backfilling the cutoff trench at approximately station 93+60 and proceeded towards the right abutment. There were several areas along the upstream slope where water was seeping from the clayey gravel zone directly above the shales. Initially the contractor only had two pumps with sumps located wherever low areas existed which resulted in the contractor chasing water from location to location. The first lift of backfill placed on the shale was 1 to 2-feet thick and in some cases was placed in standing water. Two test pits were dug between station 91+00 and 92+00 to observe the fill-shale contact and the overall backfill condition. They indicated a zone of very soft wet fill existed at the shale contact, varying in thickness from 1/4 inch to 1 inch. The contractor was required to remove and replace this unacceptable lift. The removal of this lift is shown in Photo 15. After these initial problems, the contractor's operation improved. In order to control lift thickness and ensure an adequate fill-shale contact, the contractor was directed to use special backfill for the initial shale covering which required 3-inch lift hand compacted. The backfill was brought up to a depth of 6 inches, then the next lift was compacted with a rubber tired loader. The seepage along the basal gravel was easily handled with additional pumps and pump pits.

Between station 90+10 and 91+10 several 5 gallon cans with the bottoms removed were used to collect the more concentrated seepage from the gravel zone directly above the bedrock. The 5-gallon cans were placed directly on bedrock and as the backfill progressed, additional cans with the tops and bottoms removed were placed on the initial cans. The water that collected in the cans was pumped out. After three layers of cans were installed (4.5 feet of head) the cans were filled with sand and the impervious fill placement continued.

The contractor was allowed to place fat clay rather than lean clay in the lower 5 to 6 feet of the cutoff trench (below the water table). The liquid limit restrictions were reportedly raised in this area because the excavation was wet and lean clay absorbs water more easily than fat clay, thus, the lean clay was thought to be more likely to become unworkable than fat clay. Prior to changing the liquid limit requirements the effect of the potentially lower shear strength on stability analyses was considered. It was determined that weaker material in this location near the top of rock at the centerline would not adversely affect the stability of the embankment.

The original specification required the pervious fill be placed in 12-inch lifts and compacted with three passes of a 5-ton vibratory roller or 4-inch lifts and compacted to 80 percent relative density with a plate vibratory compactor. The three passes with a vibratory roller were only achieving 75 to 80 percent relative density and the 80 percent relative density was extremely difficult to obtain using a hand compactor. Therefore, the compaction requirements for the plate vibratory compactor were reduced to 70 percent relative density for the pervious fill placement.



e. Abutments. Originally, only that portion of the excavation and foundation treatment within 75 feet of the cutoff trench and within the limits of the Stage I embankment was to be performed under Stage I. The excavation and foundation treatment was similar for both abutments. The excavation was performed with backhoes and presplit blasting to minimize damage to the rock surface. Dental concrete was used to fill joints along the excavated limestone slopes and gravity grouting was used to fill the joints along the horizontal ledges. In many places formed concrete was placed against the irregular bedrock face in order to achieve the desired slope and to seal the bedding planes and vertical joints. When the contractor began working on the left abutment it became evident the weathering extended further into the abutments than originally anticipated, therefore requiring additional excavation into the abutment to reach sound rock. Knowing the right abutment was steeper and more weathered, it became necessary to modify the contract and extend the right abutment excavation above the original limits of the top of the embankment (elevation 887). Modification 2 required the overburden and rock be excavated between station 86+00 and 87+40 above elevation 887 upstream and downstream of the cutoff trench within the final embankment limits. This additional excavation provided an adequate workbench for drilling the Bethany Falls limestone and flattened the hazardous near vertical slopes in the Winterset limestone. After the upper right abutment was excavated, the shale formations were covered with a 3-foot minimum layer of soil to protect it from drying out prior to Stage II embankment placement.

The Middle Creek limestone was present in both abutments but was more predominate on the right abutment. This formation was only about 1.5-feet thick and consisted of three distinct beds, two limestone beds with a thin shale bed in between. The upper layer of limestone was extremely jointed and had the appearance of precut limestone blocks thus requiring special care during the foundation treatment adjacent to the random and impervious zones. In order to minimize the disturbance of the rock to remain in-place, the final rock excavation was performed by equipment located on the fill not the bedrock. The three beds were stair stepped and cut to a relatively smooth vertical face. See Photos 18 and 19. The existing joints along the face of the slope were mortar treated and the joints on the top limestone surface were cleaned with air jetting and filled with grout. No heavy equipment was allowed over the Middle Creek formation until 2 feet of special backfill had been placed over it.

The Sniabar limestone was presplit in an effort to make a smoother face to compact fill against and to remove rock with solution cavities. However, on the right abutment the blasting badly fractured the formation resulting in the need for additional foundation treatment. The existing joints were opened and the upper ledge was lifted and broken up. The final treatment of the formation included the removal of upper ledge which caused undercutting of the underlying shales at various locations. The loose material was removed from the fractured face and dental concrete was placed against the irregular surface. Formed concrete was to be placed against the formation where shale undercutting occurred. The initial concrete placed against the Sniabar within the upstream random zone was placed too stiff and was not formed or vibrated. The concrete was simply dumped on the slope and towelled smooth. The contractor was required to remove and replace this concrete due to the voids and the inadequate bonding between the rock and the concrete.

f. Right Embankment. The first embankment to be placed in Stage I was on the right side of the river and was completed in 1980. Prior to any fill placement the diversion channel was excavated and the river was straightened to allow for construction of the embankment on both sides of the river. The Stage I right embankment included the upstream berm and random zones which were placed to elevation 887 and tied into the abutment at the top of the Bethany Falls limestone. It also included a small portion of impervious and downstream random zones. A plan view at the end of Stage I is shown on Plate 17 and the embankment sections are shown on Plates 23 and 24. Photo 22 shows an overall view of the right embankment. The contractor obtained the majority of the material for the right embankment from the nearby low lying borrow areas along the river which created considerable difficulty later in the contract. As previously discussed, the low lying borrow areas were the best source of the lean clay and should have been saved for the impervious zone (LL<55) in the conduit section and the stockpile. However, the contractor elected to place the lean clay in the random (LL<60) and berm (LL-unspecified) zones.

Because several bedrock zones along the right abutment had open joints or were highly fractured a 10-foot zone of clayey gravel was placed directly against the abutment to act as a filter. As the fill placement progressed the supply of clayey gravel diminished. At which time the clayey gravel was only placed against the jointed and broken up limestone formation.

g. Left Embankment. The conduit structure was completed in the fall of 1980 enabling placement of the adjacent embankment upstream of the centerline to begin. Embankment placement downstream of the centerline did not begin until the following spring, and that portion to the right of the conduit section until after removal of the sewer. The conduit section zoning extends from station 97+00 to 101+00 and consists of impervious material with the exception of the pervious blanket and inclined drain. See Plate 24. The internal zoning between station 101+00 and the left abutment originally included both berm and random zones. The contract was modified to change the upstream and downstream berm fill to random fill between stations 101+00 and 102+00. This allowed better utilization of available material, provided a better contact with the limestone surface and eliminated the need for separate zones in an isolated area.

The embankment was brought up evenly on both sides of the conduit. Downstream of the inclined pervious drain, the 3-foot thick pervious blanket was placed in contact with the foundation along the outlet channel excavation and around the conduit. Initially, the contractor had some difficulty obtaining relative density even after the requirements had been lowered to 70 percent. But once an adequate amount of water was placed in front of the compaction equipment, meeting the compaction requirements became easier.

Initially the impervious material (LL<55) for the conduit section was taken from the remaining low lying borrow areas along the river. As the impervious fill placement progressed, the contractor began to deplete the easily identified lean clay sources and had to utilize material from the higher borrow areas upstream of Longview Road. As previously discussed, the material in the higher borrow areas consisted of intermixed lean and fat clay which were difficult to distinguish visually without testing. The contractor did not have an adequate method of classifying and identifying the suitable

impervious material at the borrow pit resulting in the continual direction by the Contracting Officer to remove unsuitable material after placement in the embankment. The problem of identifying suitable material prior to its placement in the embankment and the need for extensive testing continued throughout the remaining fill placement and eventually was the basis for a major claim. Due to the contractor's mismanagement of the borrow areas, there was an insufficient amount of impervious material remaining which could be practically excavated. Therefore, the downstream portion of the embankment was topped out at elevation 867 and the upstream portion at elevation 870 even though the original plan called for a small portion of the upstream left embankment to extend to elevation 887.

h. Impervious Stockpile. As part of Stage I an impervious stockpile was constructed downstream in order to ensure the Stage II contractor a ready source of impervious after closure (Photo 25). This was necessary since the majority of the impervious (LL<55) material was located in the low lying borrow areas along the river which could periodically become inundated after closure. The contractor began working on the impervious stockpile very early in the contract utilizing the majority of the required overburden excavation from the outlet works and cutoff trench. Placement of stockpile material continued off and on throughout the length of the contract. As discussed previously, the contractor misused most of the easily accessible impervious borrow material and had to use the upper borrow areas where the impervious material was not easily identifiable. As with the impervious zone for the left embankment, extensive testing was performed both at the borrow area and in the stockpile to ensure the material placed had a liquid limit less than 55. Any material found which did not meet these requirements was removed and replaced. The specification for the gravel percentage in the stockpile was not specific and the contractor was allowed to utilize lean clay with more than 5 percent gravel. In the spring of 1982, it was discovered the impervious stockpile was located too far upstream (approximately 120 feet) within the Stage II embankment limits. The contractor was required to remove and relocate about 30,000 cubic yards of material so that it would not interfere with the Stage II embankment. The original stockpile was to be 200,000 cubic yards; however the final stockpile was actually only 170,000 cubic yards. This quantity was reduced at the end of the contract because there was very little impervious material (LL<55) remaining in the designated Stage I borrow areas. This shortage of impervious was basically due to the contractor's mismanagement of the borrow areas as previously discussed.

i. Modifications. A total of 60 modifications were made to Stage I contract. Only the following 16 pertained to the construction of the embankment.

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| No. 2  | Required additional overburden and rock excavation on the right abutment above elevation 887 upstream and downstream of the cutoff trench within the embankment toe trace. |
| No. 6  | Changed the sewer alignment from the left side of stilling basin to the right side.  |
| No. 14 | Required the excavation and backfill of test pit in cutoff trench at station 90+70+.   |

- No. 16 Required 1.) The removal of the slide material near O.W. station 52+00 and all left abutment overburden to rock downstream of the cutoff trench to the embankment toe 2.) sewer excavation adjacent to the stilling basin be backfilled with excavated overburden.
- No. 17 Required the removal of additional limestone at station 102+98.
- No. 19 Required two presplit blasting test section in the Sniabar limestone on the left abutment.
- No. 20 Change the outlet channel alignment to miss existing powerline guy wires.
- No. 22 Required the removal of two overhangs in the Bethany Falls limestone on the right abutment approximately 125-feet and 147-feet upstream of the dam axis.
- No. 25 Required the removal of overhang and the adjacent vertical face of Bethany Falls limestone at station 87+35<sub>+</sub>, 90-feet <sub>+</sub> upstream of the dam axis.
- No. 26 Required removal of fractured rock and resetting nipples for grouting in the cutoff trench between station 89+50 and 88+90.
- No. 33 Required the Sniabar limestone upstream of the cutoff trench be fractured and removed using a non-explosive demolition agent on a 1V on 1H slope.
- No. 40 Required the additional rock excavation of the Sniabar limestone in Modification 33 be extended downstream through the cutoff trench using the same non-explosive demolition agent.
- No. 41 Changed the upstream and downstream internal embankment zoning from berm fill to random fill between station 101+00 and 102+00.
- No. 46 Provided additional compensation to the contractor for slower rate of placement during curtain grouting due to a tighter rock formation.
- No. 48 Required additional presplit and production shooting of Bethany Falls limestone upstream of cutoff trench at approximate station 87+75.
- No. 55 Required the additional excavation of the unsuitable material below the bottom of the existing 48-inch sewerline and backfilling excavation with impervious fill.

8-03. Stage I Embankment Analysis. During construction of the Stage I left embankment, high pore pressures (up to about 75 percent response) were

measured in the lower part of the embankment primarily in the cutoff trench, sewer trench and conduit excavation backfills. When overflowing water levels and quick recoveries after pumping were found in the protective casing of several devices, an extensive investigation was undertaken to determine the extent of the high pore pressures and the adequacy, including stability, of the embankment. The investigation involved undisturbed sampling and testing, inspection of in-place embankment material from two large diameter auger holes OH-1 and OH-2, installation of additional instrumentation to monitor pore pressures and slope movement, and dye injection and pump tests. The investigation revealed high pore pressures in excess of 50% in the lower half of the embankment. Equipment induced shear planes or other fractures or surfaces discovered during the investigation were determined to be discontinuous. It was determined that gas escaping from the open tube devices was not from anaerobic decomposition in the fill but was apparently natural gas from deposits in the foundation rock, or might possibly have originated from gas trapped in the low lying borrow deposit. The results of the gas sample analysis are provided in Enclosure 2. Slope stability studies conducted with both total and effective stress (using measured pore pressures) analyses showed an adequate safety factor for the embankment slopes. The steepest existing slope, 1V on 3H, was found to have a safety factor of 1.8 and a safety factor of greater than 2 was found for the embankment constructed to the construction halt elevation 890. It was further concluded the embankment should perform satisfactorily as a water retention structure. A more detailed description of the investigation and its findings are provided in the "Analysis of Stage I Embankment" published in April 1983. An updated version of the write-up only is included in this report as Enclosure 1.

8-04. Stage II Contract. The Stage II contract, DACW41-82-C-0204, was awarded in August 1982 to the second low bidder, W.A. Ellis Construction Company for the amount of \$9,181,143.00, after the low bidder, Jack B. Anglin Company claimed an error and withdrew his bid. Construction began in August 1982 and was completed in September 1985. This contract included the spillway excavation, diversion and closure, completion of the embankment, construction of the service bridge, and the construction of the access and service roads. Because of the experiences in the Stage I contract and the concerns for the embankment stability, considerable effort was expended in revising and tightening the specifications and investigating the quantities of specific types of fill material available in the borrow. Based on these quantities and stability considerations, liquid limit requirements were established in detail for the embankment zones. A 6-month construction halt was scheduled in the Stage II contract when the embankment reached elevation 890. This construction halt elevation was originally specified in the DM as elevation 915; however, the construction halt was later changed because of the high pore pressures which developed in the left embankment, the actual timing of the 6-month construction halt and the relative fill quantities involved before and after the halt. In addition to the construction halt, fill placement rates were established which limited placement to 5 feet per 7-day period and no more than two lifts of fill in any 24-hour period.

a. Stage II Borrow Study. During the preparation of the Stage II plans and specifications an extensive re-analysis of the Stage II borrow area was completed. Additional drilling, sampling and testing was necessary to reanalyze the borrow area to assure that sufficient quantities of the

necessary liquid limit material were available to complete the Stage II embankment. The liquid limit requirements in the embankment zones were adjusted consistent with shear strength requirements to make best use of the material available in the borrow.

A total of 59 additional borings were completed in the west borrow area located west of old Raytown Road and 8 borings were completed in the east borrow area located south of old Longview Road. (See Plate 3.) All borings were 6-inch drive borings with a jar sample taken every 2 feet. Liquid limit, plastic limit and moisture tests were conducted to classify the material. An additional 421 liquid limit tests were completed on samples from the west borrow and 87 were completed on samples from the east borrow. The specifications called for additional testing by the contractor and placed responsibility on him to determine proper disposition of materials.

b. Diversion and Closure. The diversion and closure plan consisted of diverting the river and constructing a cofferdam in two phases. The Phase I cofferdam was to be constructed after diversion in the upstream closure area berm and random zones to elevation 878. The Phase II cofferdam was to be constructed downstream of the Phase I cofferdam and consisted of berm, random and impervious fill including backfill of the cutoff trench to elevation 887. The diversion and closure plan including sections of the Phase I and II cofferdam are shown on Plate 19.

The cofferdam heights were established in the DM based on routing various floods through the lake with the outlet works gates completely opened. The DM recommended the Phase I cofferdam height to be elevation 867 based on the October 1949 flood (fourth highest flood) which would create a lake of elevation 862.5. However, comments from MRD resulted in the height of the Phase I cofferdam being increased to elevation 878 which would protect against the second largest flood (September 1961) or lake elevation 874.4. This change was made because of the high degree of development downstream and the potential damage which could occur if the cofferdam were to fail. The height of the Phase II cofferdam, elevation 887, was designed to protect against the record flood of July 1951 which would have resulted in a lake elevation 882.3.

The contract required that diversion of the river be made between June 15, 1983 and August 1, 1983 and the Phase II cofferdam be completed by December 15, 1983. Prior to diversion the contractor had to complete the following items:

1. The left embankment cofferdam to elevation 887 as originally included in Stage I.
2. The outlet channel, including: placing bedding and riprap, grouting riprap, constructing access walk and stairs; and the removal of the protection dike adjacent to the outlet channel.
3. Final approach channel excavation.
4. The stilling basin inspection.

Within the contract there was also a list of optional items which could be completed prior to diversion. Of this list, the contractor only completed the placement of the pervious blanket and the overlying 3 feet of the corresponding embankment zones on the right side of the river.

The contractor diverted the river and began closure operations on June 16, 1983. The diversion and closure operation was performed in the following sequence:

1. The protective dike downstream of the stilling basin across the outlet channel was excavated.
2. On June 16, construction of the upstream diversion dike began and the river was diverted through the outlet works after the approach channel plug was removed.
3. The remainder of the upstream diversion dike was rapidly placed to elevation 823.
4. A portion of the fill for service road "D" between station 23+60 and 24+70 was constructed to elevation 810 to serve as a downstream cofferdam.
5. Within the Phase I cofferdam area, the material from the river channel was excavated and foundation preparation was performed on the exposed bedrock surfaces.
6. The Phase I cofferdam was rapidly constructed of berm and random material to elevation 878. Aerial views of this phase is shown in Photos 37 and 38.
7. The cutoff trench in the closure area which was not completed during Stage I (between approximately stations 93+45 and 95+80) was excavated to final grade (excavation for grouting and grouting operations were performed during Stage I) and foundation preparation was performed. The foundation treatment and fill placement in the cutoff trench closure section is shown in Photo 39.
8. The next step was to construct the Phase II cofferdam to elevation 887 before December 15, 1983. However, since diversion was made fairly early and the Phase I closure operations had gone rather quickly, the contractor elected to bring the remaining embankment up all at once rather than just the Phase II cofferdam. This fill placement is shown in Photos 43 and 46.

Shortly after diversion, heavy rains caused the Little Blue River to overtop the Phase I cofferdam which, at the time, was only a few feet above the existing ground surface adjacent to the old river channel. Some embankment material was washed away and a considerable amount of cleanup was required. The contract was modified to provide for the flood damage repair which restored the permanent work to the field conditions which existed prior to the flood damage.

Within the closure section, the pervious blanket was placed in direct contact with the foundation after the river channel was cleaned up, while the river channel downstream of embankment was backfilled with channel fill. Photo 41 shows the pervious fill placement in the old river channel and Photo 42 shows the tie-in between the adjacent pervious blanket and the pervious fill in the old river channel. The embankment toe detail within the river channel is shown on Plate 25. Downstream between station 91+00 and 93+50 an old river meander was discovered and the contract had to be modified (Modification 12) to require the removal of the debris and the low strength material. This excavation was then backfilled to the approximate adjacent ground surface with gravelly random material. The pervious blanket was then placed over the backfill rather than the foundation material.

By early November 1983, the upstream Phase II cofferdam was at the required elevation 887 while the downstream embankment averaged about elevation 880. Throughout November, adverse weather conditions prevailed and it appeared there would not be enough good working days remaining in the 1983 construction season to bring the embankment to the required construction halt elevation 890. The contractor requested the 6-month construction halt elevation be changed to elevation 880+ beginning December 1983. The Government approved the request but required the contractor do the following items prior to the construction halt: level up the embankment at station 93+00; raise the abutment contacts to provide positive drainage; level and seal the upstream cofferdam; provide erosion control for right abutment drainage ditch; and cover all pervious material with 1-foot of cover similar to the adjacent embankment material.

c. Embankment. The six month construction halt ended on June 1, 1984. The contractor had requested an early start-up date, however this was rejected because of continued high pore pressures in the embankment. In early June, heavy rains caused a lake to form which peaked at elevation 865. Although the borrow areas were inundated and numerous gullies formed on the embankment, no major damage or abnormal seepage occurred. After the high pool, the contractor removed the 1-foot impervious cover from the pervious drain and began placing fill on the downstream side. The contractor continued placing more fill on the downstream side of the embankment until the pervious drain became the highest part of the embankment and positive drainage away from the pervious drain was established. The rate of placement for fill above elevation 867 was restricted to only 5 feet of compacted fill in a 7-day period and no more than two lifts of fill in any 24-hour period. The embankment was topped out in September 1984.

d. Spillway. The 200-foot wide spillway located on the left abutment has a crest elevation 911.3 which is 2.3-feet above full pool and has a predicted use of once in 200 years. The left abutment location was selected in lieu of the right abutment due to the reduced amount of required excavation. The contractor began excavating the spillway after making diversion. The entire 550-foot length of spillway excavation was overburden consisting of relatively high liquid limit fat clay with a maximum depth of 14 feet and 1V on 6H side slopes. The contractor utilized the excavated overburden as road fill, berm or random fill. The spillway was over excavated 9 inches and brought back to grade with topsoil.



e. Service Bridge. The service bridge as shown on Plates 33 and 34 was constructed when the embankment fill reached elevation 899 (bridge deck elevation). As the fill placement progressed, the embankment began moving outward causing the service bridge expansion joints to close. Photo 63 shows a closed expansion joint and the tilted rocker. Daily monitoring of the service bridge expansion joints began in early September 1984. The movement averaged about .05-inches per day until there was concrete to concrete contact at all three expansion joints. This movement began spalling the concrete at the bridge abutment expansion joint. The joints were not repaired until the Spring 1990. A separate contract was awarded to restore the expansion joints at the abutment and the pier. Plate 34A shows the service bridge after the expansion joint repair.

f. Clay Blanket. A clay blanket was placed on the right abutment as shown on Plate 30. The clay blanket was placed between elevations 848 and 898 to control seepage thru the Bethany Falls and the Winters limestone. To utilize excess rock from the required excavations, rockfill was placed against the bedrock formations below elevation 848. Rockfill was also used to protect the upstream slope of the clay blanket (see Photo 47). Once the rockfill sources were depleted, the contract was modified (modification 26) to require 9-inch bedding and 18-inch riprap be placed between elevation 884 to 898 on the remaining face of the clay blanket slope. The complete right abutment clay blanket can be seen in Photo 72.

g. Stage II - Modifications. A total of 38 modifications were made to Stage II contract. Only the following 16 pertained to the construction of the embankment.

- |        |   |
|--------|---|
| No. 1  | Required that all material in the impervious stockpile be reserved for use after closure.   |
| No. 8  | Provided payment for the rock excavation overrun.   |
| No. 10 | Required the repair of flood damage which occurred June 28, 1983 when heavy rains caused the Little Blue River to overtop the partially completed Phase I cofferdam, and flood the old river channel.   |
| No. 11 | Required the construction of a concrete headwall at the west end of the grouted riprap drainage ditch adjacent to service road "C".   |
| No. 12 | Required the area of the embankment between station 91+00 and 93+50 (downstream of the centerline which had been excavated to remove debris and low strength material from the old slough) be backfilled with gravelly random material to approximately natural ground prior to placing the pervious blanket. |
| No. 13 | Required: The open tube piezometers PPE 99-13, PPE 97-13, PPE 97-14, PPE 99-10, PPE 89-2, PPE 89-3, PPE 93-10A and inclinometers I-97-1, and I-99-2 be extended thru the embankment; PPE 93-4 be changed from a pneumatic piezometer to an open tube piezometer; the location of PPE 93-4A be                 |

changed from station 93+00 R0+40U to station 93+00 R 2+30U; the construction of tubing ditches and the extension of tubing for 11 pneumatic piezometers; the extensions for PPE 90-1, PPE 90-2, and PPE 93-10 be deleted; and steel protective casing be installed for 9 new piezometers.

- No. 14            Changed the downstream "V" grouted gutter to a 4-foot wide flat bottom grouted toe gutter between station 87+00 and 91+00.
- No. 17            Required the borrow areas below elevation 891 be cleared completely except for the area west of Raytown Road and south of Pittenger Road; and the small area located southeast of the river immediately east of Raytown Road be cleared only to the extent necessary to obtain material.
- No. 20            Provided four lean concrete cutoff walls across the 4-foot flat bottom toe gutter on the right abutment.
- No. 22            Required: the extension of piezometers PPE 92-1, PPE 95-1, and PPE 97-16 thru the embankment and the installation of steel protective casing for each; the construction of tubing ditches and the installation of tubing for piezometers PPE 97-17, PPE 98-1, PPE 99-17, PPE 99-16.
- No. 23            Provided payment for foundation clean up overrun.
- No. 25            Required the following changes on the instrumentation monuments: changed concrete compression strength from 4000 psi to 5000 psi in 28 days; changed surface sealer from 2-inch thickness of black roofing cement to 1-inch thickness of Dow Corning 100% Silcon Rubber 795 Construction Sealant; changed rebar from 2-#4 rebars and 4-#8 rebars to 6-#4 vertical rebars equally spaced and #3 rebar circumferential horizontal ties on 8-inch centers.
- No. 26            Required 9-inch bedding and 18-inch riprap be placed on the face of the impervious clay blanket above elevation 884.
- No. 29            Required the repair of flood damage that occurred in May 1983 when rains caused the Little Blue River to overflow its banks and damage the pervious zone between stations 91+00 and 93+50.
- No. 30            Provide compensation for grouted riprap overrun.
- No. 32            Deleted the requirements            toe gutter to elevation 887+ on the upstream side of the dam near the left abutment and other minor changes.

8-05. Embankment Materials Inspection and Testing. An Excavation, Embankment, and Grouting Manual and a Field and Laboratory Testing Manual were prepared for both stages. These manuals were based on the plans and specifications and were intended as a technical guide to the Government

Inspectors and the Resident Engineer. They provided a means to better monitor the contractors work and included instructions pertaining to inspection requirements. The embankment manual also served to further explain the intent of the specification to the field inspectors. The Government inspection staff was present to witness all pertinent operation procedures and to conduct field tests. Design engineering personnel made periodic inspections, particularly during critical operations such as initial excavations, exposure of foundation surfaces, initial fill placement, diversion, and instrumentation installation.

a. Field Control Tests. Field control tests were performed to monitor moisture content, gradation, compacted density, and classification of fill material to ensure the specification requirements were satisfied. These tests were performed by the Government after the fill was compacted. The contractor was responsible for all sampling and testing to assure that materials met specified requirements prior to compaction. Monthly summaries of the field control tests were sent through Construction Division to Engineering Division and reported to MRD. Tests were performed on all material types.

(1) Impervious, Random and Berm. The field control testing for the impervious, random and berm zones consisted primarily of liquid limit and field compaction determinations. The sand cone method was used for determining the compacted density. The maximum density and optimum water content were determined in accordance with EM 1110-2-1906, Appendix VI, Standard Compaction Tests. A family of compaction curves was developed for each material source. Subsequent field tests then relied on a one-point compaction test to determine which compaction curve was appropriate. Although no fill density was specified, it was required that the fill be compacted to a minimum of 95 percent of that obtained from the standard effort compaction test. Experience had indicated that the specified moisture content and number or roller passes would achieve these densities. If the desired compaction was not achieved additional rolling was required. Due to the previously discussed problems with liquid limits on this project, liquid limits were performed on all field control samples to check compliance of the material in the various zones. If Government tests showed noncompliance with the specifications it was the contractor's responsibility to do additional testing to define limits of the "out of spec" area and to remove and replace it. The area was then rechecked by the Government.

(2) Pervious. The pervious field control testing consisted of a combination of gradation testing and in place relative density determinations. The relative density value represents a density ratio defined as the extent compaction has progressed from the loosest state toward the densest state. The minimum density was determined by placing the material dry in its loosest practical state. The maximum density was then found by vibrating the sample at a given frequency for a set time period. The detailed procedure was presented in EM 1110-2-1906 (10 May 1965).

b. Record Control Tests. Record control samples were obtained periodically from the impervious, random and berm zones. They served the purpose of checking the physical properties of the materials actually used in the dam against those properties assumed for design. Schedules of the record control sample locations and testing are shown on Plates 50 and 51 respectively. Most of the record control samples consisted of one sack and

one hand trimmed undisturbed sample 6-inches in diameter by 6-inches long (Photo 12). These samples were sent to the Missouri River Division laboratories for testing. This testing included compaction, Atterberg limits, gradations including hydrometer, moisture content, density, and shear strength determinations. Summaries of these test results are provided on Plates 52 through 85. While the actual test results are provided on half-size plates as Supplement A. The inspection staff performed standard field control tests at each record control test location. The duplication of testing by the Division laboratory and the field laboratory served to check the accuracy of the field results. A comparison between laboratories can be made by reviewing the compaction test summaries presented on Plates 79 through 85. There were some substantial variations between the laboratories in-place moisture content, compacted density, optimum moisture content and maximum dry density values. The majority of these variations are probably due to minor material changes and slightly different laboratory procedures or equipment. There is, however, a very interesting consistent variation between the laboratories' optimum moisture content results. As shown on Plates 80 and 84, the field laboratory's optimum moisture content was consistently higher than the Division laboratory's results. All of the embankment material was placed within the specified compaction requirements based on the field laboratory's standard Proctor compaction test results (95 percent maximum dry density at a moisture content between 2 percent below and 3 percent above optimum). According to the graphs on Plates 82 and 85, 26 percent of the Stage I field tests and 20 percent of the Stage II field test were outside the specified moisture range based on the field laboratory's standard Proctor test results. All of this material was supposedly removed or reworked to meet the specifications. However, these plates also show that 54 percent of the Stage I record control tests and 14 percent of the Stage II record control tests were outside the required moisture range based on the Missouri River Division laboratory's standard Proctor test results. The Division laboratory results are believed to be accurate and indicate some of the embankment material may have been placed 3 to 6 percent above optimum moisture which could have contributed to the high embankment pore pressure that developed during construction.

## CHAPTER 9

### OPERATIONAL NOTES

#### 9-01. Embankment.

a. Upstream Slope. During initial filling some erosion and undercutting of the riprap occurred on the 1 on 4 slope of the conduit section. Overall, the 18-inch riprap has held up relatively well with only some minor breakdown along the waterline at multipurpose pool elevation. During the high pool in October 1986 as shown in Photo 70, the water was near the top of the lower riprap (elevation 898). Wave action displaced the riprap leaving the bedding exposed between stations 89+00 and 92+00. In August 1989, this 300 foot section was repaired with the same Bethany Falls limestone that was used for the emergency rock stockpiles. During the May 1990 record pool (elevation 897.2) no damage was caused to the lower riprap or grassed slopes because the pool rose and fell rapidly.

b. Crest. The crest road remains in good condition except for some longitudinal cracking along the downstream shoulder caused by lack of lateral support along the pavement edge. Shortly after construction, the upstream edge of the road near the right abutment grouted gutter ravelled back past the guard rail. In 1989, the edge of the road had to be repaired to prevent any undermining of the grouted gutter. After completion of the embankment, a low area in the dike on the left abutment at approximately station 104+00 was noted. This low area was approximately elevation 925.2, 1.4 feet below the design elevation of 926.6. During the spring 1990, this low area was filled and brought up to the design elevation.

c. Downstream Slope. Since completion of the embankment, the grass cover has become established with only a few sparse areas remaining (Photo 74). There are several areas along the upper interceptor ditch which do not drain properly, causing overtopping of the ditch and the formations of erosion gullies. The lower interceptor ditch at the entrance to the left abutment grouted gutter is too low causing water to pond in the interceptor ditch. This area and a few other areas along both the upper and lower interceptor ditches have been filled to provide proper drainage. Shortly after construction, erosion control fabric was installed in the toe ditch as shown in Photo 60. Since then some silting of the ditch has occurred due to runoff from the embankment and Foreman Road. After initial filling, the sand blanket between station 91+00 and 95+50 began to seep more than initially expected. This is believed to be caused by abutment seepage. See the paragraph below. During construction, the bedding covering the sand blanket was contaminated with fines washing down the slope prior to establishing grass cover. The contractor only removed and replaced the upper 6-inches of bedding material. In 1987, drains (6-inch perforated pipe) were installed through the bedding in order to reduce the head of water exiting the sand blanket. This drain installation can be seen in Photos 65 and 66. Although all drains flow, this had no effect on the piezometer level in the sand blanket. Flow from the drains and excess seepage have caused a maintenance problem in this area. The installation of a french drain between the sand blanket and the toe ditch has been recommended to monitor the seepage and alleviate the maintenance problem.

d. Abutments. Since initial filling high piezometric levels have existed in the right abutment bedrock upstream and downstream of the grout curtain. The pressurized zones appeared to be limited to the Pleasanton Zone A, B, and Middle Creek limestone/Hushpuckney shale. The seepage is probably making its way through the grout curtain, however, with the Pleasanton Zones A and B in direct contact with the upstream rockfill zone, the seepage could also be coming around the end of the grout curtain. The seepage in any case is being intercepted by the sand blanket which extends up the abutment slope. It appears the sand blanket is adequately controlling the seepage and no other problems have developed due to those high pressures. (See previous discussion in Chapter 7.) Since completion of the dam, numerous seeps from the Winterset limestone along the right abutment have caused wet areas on the impervious clay blanket. Although this is not a dam safety concern it became a maintenance problem. In 1987, a french drain was installed to dry up the area (Photo 67). However, the french drain did not extend into the draw far enough and does not intercept all of the drainage. Therefore, some wet areas in the impervious blanket still exist. The downstream right abutment grouted gutter has held up well since end of construction because it was constructed with concrete cutoff walls every 50 feet. These cutoff walls were added as a modification during construction which widen the gutter to intercept a natural draw coming down the right abutment. The downstream left abutment grouted gutter was not constructed with cutoff walls and consequently began undermining. In September 1989, concrete head walls were constructed on the left and right abutment grouted gutter where the interceptor ditches flow into the grouted gutters. Also, concrete cutoff walls were installed about every 75 feet along the left abutment grouted gutter to prevent the undermining.

9-02. Emergency Rock Stockpile. The construction of two emergency rock stockpiles were completed in March 1990, on both abutments. Photo 75 shows the completed left abutment emergency rock stockpile. The stockpiles consist of 8,084 tons of Type "A" rock 1,826 tons on the left abutment and 6,259 tons on the right abutment, and 3,231 tones of Type "B" rock, 747 tons on the left abutment and 2,484 tons on the right abutment. Rock for the stockpiles was produced from the 16-foot ledge of Bethany Falls limestone in Carefree Quarries on the northeast corner of the intersection of Highway 291 and Kentucky Road in Sugar Creek, Missouri. Listed below is the specified rock gradations:

Type "A" (Riprap)

<u>Weight</u>	<u>Specified Percent of Weight Lighter Than</u>
760#	100
570#	75-95
190#	30-50
55#	0-2

Type "B" (Bedding)

<u>Sieve Size</u>	<u>Specified Percent By Weight Passing</u>
6"	100
4"	82-95
2"	62-83
1"	32-64
3/8"	0-20

ENCLOSURES

ENCLOSURES



ENCLOSURE 1

LONGVIEW DAM  
ANALYSIS OF STAGE I EMBANKMENT

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## I. Introduction

1. The purpose of this report is to present the results of a geotechnical investigation of the Longview Stage I embankment. At the completion of embankment construction, high pore pressures were known to exist in the lower part of the embankment. However, when overflowing water levels and quick recoveries following pumping were found in the protective casing of several devices an investigation was undertaken to determine the extent of the high pore pressures and the adequacy, including stability, of the embankment.

2. Longview Dam is constructed on the Little Blue River in southeast Kansas City, Missouri. The dam, an earthfill zoned embankment, was built in two stages. The Stage I contract, completed in the fall of 1982 (see Photos 32 and 33), consisted of foundation excavation and preparation including grouting; construction of the intake tower, conduit, relocated sewer, and stilling basin; excavation of the outlet channel; and placement of a portion of the embankment on both sides of the river. Stage II includes diversion and closure, and completion of the embankment.

3. The Stage I contract began in late summer of 1979 with the outlet works excavation. The right embankment and cutoff trench were completed during the 1980 construction season. By fall of 1980 the outlet works conduit was completed which enabled placement of impervious backfill upstream of centerline. In the spring of 1981, the outlet works backfill downstream of centerline began. Sewer diversion through the conduit took place in July and was followed by excavation and removal of the old sewer line and backfill of the excavation. Heavy rainfall throughout the spring and summer delayed work, but the fill had been raised above the conduit, El. 822+, by the end of August. By mid-November the downstream embankment was topped out to El. 867 and the upstream cofferdam to El. 870. The finished height of the Stage I cofferdam was reduced from El. 887 because of a shortage of impervious borrow for the Stage I contract.

## II. Embankment Pore Pressures, End-of-Construction

4. During the Stage I construction period, 19 pore pressure devices were installed in the left embankment and foundation area (see table 1). Five of these, PPE97-1, PPE97-6, PPE99-2, PPE99-5, and PPE99-6, whose tips were located in the embankment clay showed pore pressure responses of 78, 68, 53, 13, and 56 percent, respectively, following the embankment construction in October 1981. PPE99-5 is located near the inclined sand drain which probably accounts for its lower response. It was concluded at the time (fall of 1981) that even though the responses were 50 percent and above, the tips were located within the backfill of the excavations and these pressures did not extend into the upper part of the embankment where they might contribute to slope instability. Pore pressures in the overburden and foundation shale were low or nonexistent, with one exception. PPF99-4 showed about 50 percent response. Its tip was located deep in the foundation shale and did not influence stability.

5. In January 1982, ice was observed at the top of the protective casing around PPE99-2. This was concluded to be either communication between the riser and the protective casing (since at that time the water level in the riser was higher than in the protective casing) or that water could have

TABLE 1  
LISTING OF INSTRUMENTATION

Number	Station	Range	Type	Installation Date	Material Type	Tip Elevation	Zone of Sand Backfill	Bottom of Protective Casing	Ground Elevation at Installation	Ground Elevation Present	Remarks
Left Embankment											
FFE97-1	97+00	325' US	Pneumatic	3 Sep 81	Embankment-Sewer Backfill	820.2	819.6-822.2	—	823.2	870 ±	dry hole (1)
FFE97-2	97+00	210' US	Pneumatic	3 Sep 81	Embankment-Sewer Backfill/Fdn Overburden	820.6	819.6-822.6	—	827.6	870 ±	dry hole (1)
FFE97-3	97+00	200' US	Open Tube	4 Sep 81	Fdn Shale	794.8	794.5-797.5	801.5	827.5	866 ±	groundwater at EL. 810.8 (2)
FFE97-4	97+00	90' US	Open Tube	25 Feb 80	Fdn Gravel	810.5	810.2-813.3	817.7	828.2	844 ±	dry hole (3)
FFE97-5	97+00	40' US	Open Tube	2 Sep 81	Fdn Shale	785.0	784.8-788.0	792.0	825.5	839 ±	no water noted (3)
FFE97-6	97+00	15' US	Pneumatic	3 Sep 81	Embankment-Cut-off Trench	807.0	805.2-809.0	—	825.2	839 ±	dry hole (1)
FFE97-8	97+00	85' DS	Open Tube	25 Feb 80	Fdn Overburden	817.8	817.4-820.4	821.0	823.4	839 ±	dry hole (3)
FFE97-9	97+00	100' DS	Pneumatic	24 Sep 81	Fdn Shal	789.9	789.5-792.5	—	836.9	839 ±	no water noted (1)
FFE97-10	97+00	120' DS	Open Tube	2 Oct 80	Fdn Gravel	805.0	803.4-808.0	812.0	825 ±	839 ±	no water noted (3)
FFE97-11	97+00	180' DS	Open Tube	22 Sep 81	Embankment-Imp./Sand Drain	824.6	824.5-827.6	831.5	834.5	839 ±	dry hole (3)
FFE97-12	97+00	200' US	Open Tube	29 Oct 82	Embankment-Injurious	853.6	853.4-855.6	866.6	869.6	869.6	.9' of rainwater in hole overnight; water added to fill riser to top
FFE97-13	97+00	190' US	Open Tube	30 Oct 82	Embankment-Injurious	839.9	839.5-842.2	867.9	869.7	869.7	dry hole; water added to fill riser to top

TABLE 1 (Cont'd)

Number	Station	Depth	Type	Installation Date	Material Type	Tip Elevation	Zone of Sand Backfill	Bottom of Protective Casing	Ground Elevation at Installation	Ground Elevation Present	Remarks
PFE97-14	97+00	195' US	Open Tube	1 Nov 82	Exhaustment- Imperious	829.7	829.2-832.0	867.7	869.7	869.7	dry hole; water added to fill riser to top
PFE97-15	97+00	202' US	2" O.T.	6 Nov 82	Exhaustment- Imperious	825.4	824.9-842.7	—	869.7	869.7	dry hole; added 12 gal. of water
PFE99-1	98+51	220' US	Pneumatic	19 Aug 81	Fdn Shale	800.0	799.5-802.5	—	825.7	872	no water noted (1)
PFE99-2	98+44	80' US	Open Tube	18 Aug 81	Exhaustment- Imperious	812.0	811.3-814.3	818.8	824.3	867 ±	dry hole; 5 ga. water added
PFE99-3	98+42	40' US	Open Tube	20 Aug 81	Fdn Shale	789.7	789.4-792.7	796.7	824.7	867 ±	no water noted (3)
PFE99-4	98+38	40' DS	Open Tube	21 Aug 81	Fdn Shale	772.8	770.8-773.9	777.9	823.9	867 ±	water encountered at sand blanket El. 811.9 ± (2)
PFE99-5	98+37	60' DS	Pneumatic	21 Aug 81	Exhaustment- Imperious	818.0	817.5-820.6	—	823.5	867 ±	no water noted (1) located near inclined sand drain
PFE99-6	99+12	220' US	Pneumatic	18 Aug 81	Exhaustment- Imperious	822.5	822.0-824.0	—	826.5	867 ±	no water noted (1)
PFE99-7	98+40	70' US	Open Tube	11 Jan 82	Fdn Overburden/ Fdn Shale	810.3	808.5-811.0	816.5	866.5	866.5	no water noted (2)
PFE99-8	98+45	80' US	Open Tube	27 Oct 82	Exhaustment- Imperious	823.8	823.0-834.2	863.8	867.0	867.0	9' of water in hole after 35 min.; water added to fill riser to top
PFE99-9	99+00	50' US	Open Tube	3 Nov 82	Exhaustment- Imperious	823.4	823.3-832.2	865.2	867.2	867.2	dry hole (4)
PFE99-10	99+00	150' DS	Open Tube	5 Nov 82	Exhaustment- Imperious	823.0	822.9-832.9	863.5	866.9	866.9	.1' water overnight (5)

TABLE 1 (Cont'd)

Number	Station	Range	Type	Installation Date	Material Type	Tip Elevation	Zone of Sand Backfill	Bottom of Protective Casing	Ground Elevation at Installation	Ground Elevation Present	Remarks
PPE99-11	98+44	85' US	2" O.T.	8 Nov 82	Embankment-Impervious	821.2	820.5-839.2	—	867.2	867.2	water in hole but depth not noted (2)
PPE99-12	98+40	6	Open Tube	12 Nov 82	Embankment-Impervious	838.6	838.6-849.0	864.0	867.0	867.0	4' of water 30 minutes after drilling (4)
PPE99-13	98+40	200' US	Open Tube	16 Nov 82	Embankment-Impervious	824.8	824.5-834.9	865.9	868.9	868.9	.1' water 30 minutes after drilling (4)
*PPE99-14	98+40	280' US	Open Tube	17 Nov 82	Embankment-Impervious	823.2	823.1-833.1	864.1	867.1	867.1	trace of water 45 minutes after drilling (4)
PPE99-15	98+35	6	Open Tube	13 Nov 82	Embankment-Impervious	823.2	823.1-832.9	864.0	867.0	867.0	dry hole (4)
*PPF100-1	99+70	200' US	Open Tube	10 Nov 82	Fdn/Overburden Fdn Shale	815.7	813.9-825.7	866.7	869.7	869.7	.5' water in hole 30 Min. after drilling (2)
PPF 61	99+85	400' US	Open Tube	13 Apr 71	Fdn Shale	805.1	804.6-808.6	809.6	842.6	851 ±	no water noted (4)
PPF 61A	99+95	400' US	Open Tube	16 Apr 71	Fdn Overburden	813.1	812.6-816.6	817.6	842.6	851 ±	no water noted (4)
PPF 359	99+99	295' US	Open Tube	26 Feb 80	Fdn Overburden/Top of Fdn Limestone	816.0	814.5-835.0	838.0	840.0	862 ±	no water noted (4) (5)
CH-1	98+00	143' US	4" O.T.	24 Nov 82	Embankment-Impervious	825.0	825.0-864.0	—	870.0	870.0	38" auger hole backfilled with compacted sand
CH-2	98+12	6	4" O.T.	6 Dec 82	Embankment-Impervious	820.0	820.0-861.8	—	867.0	867.0	38" auger hole backfilled with compacted sand

TABLE 1 (Cont'd)

Number	Station	Range	Type	Installation Date	Material Type	Tip Elevation	Zone of Sand Backfill	Bottom of Protective Casing	Ground Elevation at Installation	Ground Elevation Present	Remarks
<b>Night Embankment</b>											
PZF 229	91+60	80' US	Open Tube	16 Dec 75	Fdn Gravel/ Fdn Shale	805.5	804.5-811.5	812.5	820.0	848 ±	Groundwater at El. 810.2 (2)
PZF 230-1	89+50	199' US	Open Tube	23 May 82	Embankment- Random	835.1	834.8-838.8	884.0	886.8	886.8	dry hole (3)
PZF 230-2	89+45	199' US	Open Tube	20 Nov 82	Embankment- Bern	862.2	862.0-866.1	884.4	886.8	886.8	dry hole (3)
PZF 230-3	89+55	199' US	Open Tube	1 Dec 82	Fdn Overburden	814.3	813.3-818.1	883.4	887.3	887.3	dry hole (3)
<b>Inclinometers</b>											
I 97-1	97+49	189' US	Inclinometer	11 Nov 82	Fdn Shale	797.8	Top of bedrock - 807.8		870 ±	870 ±	water encountered at El. 827
I 99-1	98+36	256' DS	Inclinometer	15 Jan 82	Fdn Shale	796.5	Top of bedrock - 800.0		867 ±	867 ±	dry hole
I 99-2	98+25	85' US	Inclinometer	20 Nov 82	Fdn Shale	797.8	Top of bedrock - 809.6		867 ±	867 ±	water encountered at El. 835.5

## Legend

PPE	Pore Pressure Device - Embankment
PFF	Pore Pressure Device - Foundation
PZE	Piezometric Device - Embankment
PZF	Piezometric Device - Foundation
OH	Observation Hole - Large Diameter (38") Auger Hole
I	Inclinometer

- (1) Pneumatic cell filter stone was saturated prior to installation and sand surrounding cell was saturated prior to sealing off cell.  
 (2) Wet installation, no additional water added to riser.  
 (3) Sand surrounding device saturated after riser installed.  
 (4) Dry installation, no water added to riser.  
 (5) Station and Range as of February 1983

\* Abandoned as of March 1983

entered the casing by a number of external sources such as rainfall, or by someone inadvertently or deliberately pouring water down the casing.

6. To determine the source of this water and to obtain additional undisturbed record control samples, PPF99-7, an open tube device, was installed in early January 1982 in the foundation overburden/foundation shale contact 10 feet downstream from PPE99-2. By the end of the month, water had filled the casing of PPF99-7. On this basis, it was concluded the water was coming from unknown external sources and not from the fill itself. Communication between the riser and the casing was not likely since the water level in the riser was 45 feet lower than in the casing. In addition, water had not been observed in the casings on other KCD projects unless the riser had overflowed. It was believed that the casing with slip joint couplings supplied with "O" rings would be watertight. In addition, it was not believed possible that pore water in an impervious fill could accumulate that fast in the casing when the usual experience is that a large amount of water must be added to open tube devices to bring them to equilibrium.

7. By fall of 1982 the devices began showing varying degrees of dissipation of pore pressure. However, in September 1982 in preparation for winter, the water levels in the casings of PPE99-2 and PPF99-7 were lowered as much as 10 feet to prevent freezing, but one week later the water had recharged to its original level in PPE99-2 and had recharged halfway in PPE99-7. In addition gas bubbles were being released from the casing water of PPE99-2. There was no known external source of water. It appeared that the protective casings and the risers were acting as separate measuring devices. The protective casings were apparently registering higher pore pressures at points higher in the embankment, possibly near the mid-height. These pressure levels apparently exceeded 50 percent response.

8. An extensive investigation was undertaken to determine: 1) the extent and magnitude of pore pressure levels in the embankment, and 2) the source of apparently excess water in the impervious zone of the embankment.

### III. Field Geotechnical Investigation

9. Starting on 25 October 1982, twelve open tube pore pressure devices were installed at various locations and depths in the left embankment between station 97+00 and station 101+00 to determine if a zone of high pore pressure did exist near the mid-height of the fill and if it did, to define its limits. All instrumentation holes were continuously sampled with a 5-inch Hvorsley Shelby tube sampler. Since it was suspected that use of drilling fluid may have caused hydraulic fracturing of the fill at PPF99-7, all succeeding holes were cleaned after each push with a 6-inch box auger and no drilling fluid was used. Some squeezing of the hole occurred after the holes had been advanced to about 30 feet. (This was also about the same depth water was encountered in several of the holes.) In taking the undisturbed sample the driller lowered the Hvorsley sampler usually until a slight seating resistance was encountered (normally .2 to .3 feet above the clean-out depth). This allowed space for any material reamed from the hole sides by the sampler. Inspection of the samples (some were sliced thin and air dried) did reveal a few cases of disturbance at the top of the sample which may have been caused by seating or heaving of the hole. Generally the quality of the undisturbed samples was excellent.

10. Drilling started with hole PPE99-6, located about 11 feet from PPF99-7. Free water was encountered while it was being drilled. Thirty-five minutes after drilling was completed, the hole had 9 feet of water in it. There was concern the fill was cracked or had been hydraulically fractured from the earlier use of drilling fluid in hole PPF99-7.

11. To investigate the lateral extent of these fractured zones dye was added to piezometers PPE99-8 and to subsequently drilled PPE97-13 on 3, 4, and 5 November. Drilling began on PPE99-11 located 5 feet from PPE99-8 on 7 November 1982. Free water was encountered in drilling this hole, however no dye was found, either in the water or in the undisturbed samples. When the water was bailed to the bottom of the hole, water levels in adjacent devices PPE99-8 and PPE99-2 were affected. The casing water level in PPE99-2 (about 6 feet from PPE99-11) dropped 3.1 feet and the riser water level in PPE99-8 dropped 8 feet. PPE97-15, located 5-1/2 feet from PPE97-13, was drilled on 6 November. No free water or dye was encountered. No trace of dye was found in any of the undisturbed samples.

12. In addition, after adding water to the top of the riser in PPE99-8 shortly after installation, the water was observed surging up and flowing out the riser. This phenomenon was not understood at the time, however it was speculated this could have been sudden movement (shearing) of the fill or could possibly be associated with the escape of gas from the riser since the fluid was very "bubbly." Gas bubbles were observed escaping from several of the devices. There was also speculation the fill might contain a shear zone or cracks from differential settlement over the conduit and the adjacent alluvium, but it was still considered more likely that this area of the fill had been hydraulically fractured. There was also concern that the gas was an indication of organic material in the fill.

13. The undisturbed samples from all holes were closely examined to determine if shear surfaces, cracks, free water, or organic material could be found. The samples usually had a tendency to break on both smooth horizontal planes (see Photo 27) and on more irregular surfaces which had a raindrop-type texture (see Photo 28), as if they had been wet at one time. However, no free water was found, except from one sample from hole PPE99-14, El. 835.3, where a slight indication of free water was found, and from one sample from PPE97-13, El. 839.3, which appeared even wetter. More samples were examined from other holes and these planes were found in most samples examined except to perhaps a lesser extent in those from near the ground surface. Some of the planes could be lift surfaces, however close spacing and orientation precluded others from being lift planes. Most of the planes did not give any indication that free water had traveled over their surfaces. The planes often seemed to be coated with silt or appeared to be a silt parting. No organic material was found in any of the samples. A high angle fracture was found in sample 8 from PPE97-14, depth 40.0' to 40.5', El. 829.2-829.7. This fracture did not extend to any adjacent holes and may have been caused by drilling and sampling procedures.

14. Two inclinometers in addition to I-99-1 (previously installed in January of 1982 on the downstream slope) were installed to determine if any significant slope movement was taking place on the temporary IV on 3H and IV on 3.5H slopes parallel to the river. To date, these devices have not shown any indication of significant movement.



15. The open tube devices with tip below a depth of about 30 feet began showing water levels either close to the top of the riser or began overflowing. In several installations (PPE99-9, PPE99-10, PPE99-11, PPE99-13, PPE99-14, and PPE99-15) the tip was installed dry. No water was added to bring the device to equilibrium. Yet, the water level in the riser rose typically 1 to 2 feet per day. PPE99-13 rose at a rate of about 9 feet per day. As noted previously, this is contrary to behavior observed previously at KCD projects. Those which overflowed, PPE97-14 PPE99-9, PPE99-10, and PPE99-13, were equipped with pressure gauges and air eliminator valves. The pressure began approaching and in some cases exceeded previously monitored pore pressures in the lower portion of the embankment. However, the air eliminator valves were later found not to be effective in completely removing gas from the devices. It was later concluded the readings were higher than the actual pore pressures since gas partially displaced the water in the riser.

16. The right embankment area was originally instrumented with only one piezometer located in the foundation gravels. When these higher pore pressures were found on the left embankment, three pore pressure devices were installed in November 1982. Their tips were located in the foundation overburden, the random fill zone, and the berm fill zone. The berm shows no evidence of pore pressure. Pore pressure response in the random zone was 30 percent, while the response in the foundation was 25 percent.

17. In order to be sure the embankment was suitable and in fact stable, it was considered prudent to auger two large diameter (38-inch) holes to allow detailed inspection of the fill (see Photo 26). The locations were selected at station 98+00, range 143 u/s and at centerline near the edge of the intersection of the 1V on 3H and 1V on 3.5H slopes. This was considered the most likely location for differential settlement cracks or for a shear zone associated with slope instability to appear. Holes OH-1 and OH-2 (see Plate 49) were drilled in ten foot increments to depths of 45 and 47 feet, El. 825 and El. 820 respectively. Temporary casing was installed to allow personnel to safely enter for inspection and sampling (both bars and hydraulically jacked 5-foot diameter undisturbed samples were taken).

18. In OH-1, water seeps were observed at 22 feet, El. 848 (see Photos 29 and 30); 24 feet, El. 846; and 34 feet, El. 836 (see Photo 31). Gas bubbles were observed forming on some of the wet surfaces. At a depth of 32 feet the air in the auger hole contained 10 percent combustible gas according to measurements from a portable gas "sniffer." Although the quantity of water coming from the seeps was not large enough to collect, after about a week the sidewalls had become wet below the seep areas. Concentric cracking also had developed and these cracks had free water on the surfaces.

19. The second hole, OH-2, was drilled on the centerline to determine if any fracturing extended through the central impervious core of the dam. Water was encountered only at 23 feet, El. 844. A thin, very silty lean clay zone was encountered at 37.3 feet, El. 829.7. The rest of the embankment material seem to be well placed and well compacted with only the usual material color changes taking place along with minor variations in stiffness. The planes or fractures were not as evident as they had been in the undisturbed Shelby tube samples. It appeared some planes on the side wall and in the cuttings (similar to ones previously observed) were in fact caused by mechanical

shearing from the auger. It is considered likely that many of planes observed in the undisturbed samples were caused by mechanical shearing from construction equipment (e.g. scraper pan, motor patrol blade, or pumping action from heavily loaded scraper tires), or could be related in some way to the drilling and sampling procedures. Similar planes were observed at Blue Springs Dam following stripping operations performed with scrapers. These planes were of limited extent (equipment width) and tended to dip only slightly from horizontal and in the direction of equipment movement. They appeared to have been caused by shearing action of the scraper pan. Their length was relatively short, generally 5 feet or less.

20. Gas samples were collected from the open tube devices after the observation of gas in the instrumentation risers and casing and after combustible gas was detected in the auger hole. Initial attempts were made with a small neoprene balloon. A local commercial lab analyzed the gas as a "50 percent methane/50 percent air" mixture. (See Enclosure 2.) Since no organic material other than undecomposed small roots was found in any drill hole the source of the methane was unexplained. The flammability of the gas in all the open tubes which had pressure gauges was checked by holding the air release valve open. In all cases where a quantity of gas had built up, the gas could be ignited (see Photo 34). Taking care to avoid dilution or contamination, two additional gas samples were taken in stainless steel "bombs" and analyzed by the Environmental Lab at the Waterways Experiment Station. These were taken from PPE99-9 and PPE99-10. Both have tips located in the impervious embankment, but PPE99-10 is isolated by the sand blanket from other suspected sources of gas such as the abutment, the foundation, and the relocated sewer. It is also isolated from the impervious core and upstream fill by the inclined sand drain.

21. Results of the WES tests on samples from both locations show the content of methane (percentage by volume) varies from 65 to 70 percent and that of carbon dioxide varies from 16 to 20 percent. An aerobic decomposition of organic material normally results in a lower methane content and a much higher carbon dioxide content than was found in the samples. The gas analysis showed that the fill was not producing methane as a result of decomposition of organic material. The results of the gas tests are provided as Enclosure 2. Analysis of water samples from the devices showed no indication of treated water or sewage. The results from the water analysis are provided in Enclosure 3. It was initially hypothesized that the gas was present in the soil voids in the low lying borrow deposit, either as a result of underlying gas fields or overflow of sewerage. It is difficult to believe the gas could escape from the foundation or the sewer, pass through the pervious zone and into the impervious fill, considering the high pore pressures present in the latter as opposed to no excess pressures in the former. However, if enough capillary tension existed in the sand fill it may be possible for the gas to pass through the pervious zone.

22. In addition to dye injection, pump tests were conducted at two of the installations (PPE97-15 and PPE99-11) which were installed with 2-inch diameter PVC risers and a 15-foot section of 2-inch slotted well screen. Each were located adjacent to piezometers to provide monitoring of drawdown. Initial pump testing drew the water level down only 12 feet, the drawdown being limited by the pump suction. This had no discernible effect on adjacent devices. Subsequent "pump testing" was actually done by bailing rather than

pumping in order to draw the water surface down farther. Pumping PPE97-15 affected the water level only in PPE97-13 (5.5 feet away). It did not affect PPE97-12 (7 feet away), and PPE97-14 at (8 feet away). Pumping PPE99-11 immediately affected the water level in PPE99-8 (5.0 feet away) and the casing water level of PPE99-2 (6.5 feet away). Casing and tip water levels in PPE99-7 (approximately 17 feet away) were not affected. This data and data from the dye injection indicate some fractures or cracks do exist over at least limited areas and may be capable of transmitting water only short distances, say less than about 10 feet.

23. Coefficients of permeability were determined. Using the results of dye injection, field falling head permeability tests conducted in conjunction with the dye injection, and recharge data from pump tests. Dye injection showed a seepage velocity of .0019 cm/sec or a  $k$  of  $10^{-3}$  cm/sec. Falling head permeability data yield a  $k$  on the order of about  $10^{-5}$  to  $10^{-6}$ ; recharge data shows a range from about  $10^{-4}$  to  $10^{-6}$ . These permeability values are also consistent with the rather rapid rise of the water level in several of the installations at station 99+00. It is difficult to believe this quantity of water could flow at this rate (about .5 gallons per day) from a compacted "impervious" fill. It can only be theorized that the presence of fractures or cracks increase the area of the drainage boundary. This allows faster collection of water produced from consolidation or pore pressure dissipation in the area surrounding the borings. Since the induced construction pore pressures obviously have only dissipated slightly these numbers can be indicative of only localized areas

#### IV. Laboratory Testing

24. The laboratory investigation consisted of classification testing of both jar and undisturbed samples; and strength and permeability testing of undisturbed samples. Jar samples were obtained both during the installation of instrumentation and from the side walls of the two large diameter auger holes. Undisturbed samples along with sack samples were obtained for record control testing during construction. Additional undisturbed samples were obtained from push samples taken both during the installation of instrumentation and from the side walls of the two large diameter auger holes. Testing was also done on a sack sample taken from a small seep area located on the IV on 3.5H slope of the left embankment.

25. Water content, Atterberg limits, and/or mechanical analysis tests were performed on representative jar samples. Specifications for the left embankment between station 97+00 and station 101+00 required a liquid limit of less than 55 and a moisture content between 2 percent below and 3 percent above the standard Proctor optimum moisture content. Very few of the liquid limit results exceeded the liquid limit requirement. A review of the field compaction data compiled as the embankment was constructed indicates a mean moisture content of 1.2 percent above optimum with a standard deviation of  $\pm 1.4$  percent. (See also paragraph 8-04.b. for discussion of moisture content vs optimum water content.)

26. Testing of undisturbed samples consisted of unconsolidated-undrained triaxial compression "Q" tests; consolidated-undrained (with pore pressure measurement during shear) triaxial compression  $\bar{R}$  tests, drained direct shear "S" tests, and constant head permeability tests. The record control samples

taken during construction had "Q", "R" and direct shear "S" tests run on each sample. At the time of this report there had been a total 14 "Q", 20 "R", 15 "S" and 16 direct shear "S" tests performed on material from the left embankment. The results of the "Q" tests confirm the design strength of  $c = .9$  tsf (see Plate 54). The effective stress "R" or "S" strength envelope developed from record control and recent testing indicated strength parameters of  $c = .15$  tsf and  $\tan \phi = .49$ . This compares with the design strength envelope of  $c = 0.0$ ,  $\tan \phi = .50$  shown on Plate 54.

27. Three horizontal constant head permeability tests were performed on samples from OH-2 with the first sample being oriented with existing planes. The two other tests were run on a silty lean clay material encountered deeper in hole. The permeability of the first sample was  $1.9 \times 10^{-7}$  cm/sec while the permeability of the two samples in the silty material was  $2.2 \times 10^{-8}$  and  $2.6 \times 10^{-8}$  cm/sec.

28. Moisture density tests were run on a sack sample obtained from a seep area on the LV on 3.5H slope at station 97+35, range 60' u/s, El. 848. Triaxial "Q" and "R" testing was done on samples remolded with a water content of 3 percent above optimum. Results from the "Q" test were slightly below design strength. Effective stress data from the "R" testing indicated a strength above the design envelope.

#### V. Analysis of Instrumentation Data

##### A. Embankment Pore Pressure Measurements

29. The pore pressure in the devices installed in the embankment during construction peaked shortly after the cessation of Stage I fill placement in November 1981 and dissipated slowly until resumption of construction. Pressure in PPE97-1, installed in the old sewer-excavation backfill, peaked at a level of almost 28 feet above the top of the embankment, El. 898, and dissipated 10 feet. Pressure in PPE99-6, installed near the foundation, upstream of centerline and left of the conduit, rose to 6 feet above the fill and dissipated 3 feet. The water level in PPE99-2, the only open tube with its tip installed in the fill, peaked at a level of 3 feet above the fill and dissipated 16 feet.

30. The twelve additional open tube pore pressure devices, installed in the left embankment area since October 1982 were all in the embankment impervious zone. Two devices (PPE97-12, PPE99-12) installed in the upper 20-30 feet of the fill showed no pore pressure. PPE97-13 (tip at about 30 feet) showed about 30 percent response.

31. Three pore pressure devices stabilized with their water level within about 10 feet or less below the top of the embankment. They include PPE99-8, PPE99-11, PPE97-15. These devices had tips at elevations of about 821 to 832. The remaining six devices (PPE97-14, PPE99-9, PPE99-10, PPE99-13, PPE99-14, and PPE99-15) had pore pressures in excess of 50 percent. All had tips in a zone from about El. 823 to 833. All were equipped with pressure gauges and air eliminator valves. Although these valves have been used successfully in the past on other KCD projects to remove gas from piezometers they have not worked well here. As long as the gas accumulated slowly and at relatively low pressures, the air eliminator system worked. However, when gas accumulated

rapidly and at higher pressures the valve did not release the gas and allowed it to build up and cause an abnormally high gauge reading making the magnitude of the pore pressure impossible to determine. When the gauges were removed a mixture of both water and gas bubbles would overflow the riser. Some of these bubbles were from gas coming out of solution as the pressure was reduced. However the earlier described phenomenon with the piezometer surging, then overflowing had occurred prior to the installation of a gauge. It is believed possible that gas was somehow trapped in the riser and had to develop a certain volume or pressure before it would rise to the top. Evidently a larger diameter riser (e.g. PPE99-11 and PPE97-15) allows gas that collects in the riser to escape more easily with less effect on the water column. It appears, therefore, that the column of water in the riser was not all water but a mixture of water and highly concentrated trapped gas bubbles.

32. This was further evidenced when gauges were initially removed from PPE99-9, PPE99-10, and PPE97-14, and a collector was fabricated to catch any overflow from the risers. The collector allowed the monitoring of both the amount of overflow and the fluctuation of the water level in the riser. The devices do not overflow at a constant rate over any given period of time. As an example, PPE99-10 overflowed 38 ounces between 14 February and 17 February. By 18 February it had overflowed an additional 18 ounces. On 1 March, the collector contained 99 ounces after just 4 days but the water level in the riser had fallen to a depth of 29 feet. The next day, 2 March, the water level had come up 10 feet to a depth of 19 feet. By 3 March it had come up an additional 5.5 feet and was rising at a rate 1/4-foot per hour. It was rechecked on 11 March and found that it had overflowed and was frozen. Fluctuations of the water level in the other riser are not as extreme, but all devices have shown fluctuations up to 2 feet within a few minutes.

33. The water level in the casings of PPE99-2, PPF99-7, PPF99-3, and PPF99-4 are also acting as separate devices. All water levels are slightly above the top of the embankment. Both PPF99-3 and PPF99-4 casings have consistently bubbled with escaping gas.

34. Based on the above data it is believed the pore pressure response in this zone between El. 820+ to 830+ is probably on the order of 50 to 60 percent.

#### B. Inclinerometers

35. Three inclinometers have been installed in the left embankment. I-99-1, installed in January 1982, is located on the downstream LV on 4H slope adjacent to the river closure section. The maximum movement indicated has been to 1/10-inch downstream and 4/10-inch riverward. Movement is toward the river where the embankment slope is LV on 3.5H. I-99-2 and I-97-1 were installed in November 1982 near the riverward slope of the embankment, upstream of centerline. I-99-2, on the LV on 3.5H slope, has indicated a maximum movement of approximately 1/10-inch in the downstream direction and essentially no movement toward the river. I-97-1 has indicated a maximum movement of approximately 1/10-inch both in the downstream direction and the riverward direction.

## VI. Slope Stability

36. Stability studies were made for the end-of-construction condition using both total stress and effective stress methods. The computer program used was STABR developed by the University of California at Berkeley which uses Bishop's modified method of analysis and a circular failure surface. The program contains a searching routine which varies both the center of the circle and the radius of the circle until the minimum safety factor is found. Critical minimum slides were checked by a hand analysis to verify the computer safety factor. Pore pressure contours developed from measured induced construction pore pressures were used for the effective stress analysis. The shear strength for the soil was determined from record control testing and from testing on undisturbed samples taken during the field investigations. The total stress analysis used "Q" strength and the effective stress analysis used drained "S" and undrained "R" strength with pore pressure measurements.

37. Two slope configurations have been studied on the left embankment: (1) the LV on 3H slope along the closure area with the embankment at El. 871 which is the existing condition, and (2) the upstream LV on 4H slope in the conduit section with the embankment completed to the construction halt, El. 890. Pore pressures used for this latter case were based on measured pore pressures and the assumption that the additional embankment load would cause a pore pressure response of 50 percent. A 50 percent response is reasonable based on previous KCD experience where pore pressure has been allowed to dissipate after a construction halt. Generally the additional response should be less after some dissipation (consolidation) has occurred.

38. Initial studies begun in December 1982 (with construction induced pore pressures reflecting the highest pressure gauge readings) showed a minimum safety factor of 1.47 for the temporary LV on 3H slope. Both a total stress and an effective stress analysis was conducted for the LV on 4H upstream slope constructed to El. 890. The minimum safety factor for the effective stress analysis was 1.23 while the minimum safety factor for the same slide for a total stress analysis resulted in a safety factor of 2.1. Later it was concluded the high pore pressures indicated by the pressure gauges were not valid because of the influence of gas and further analyses were conducted with the pore pressure contours believed to be more representative of actual conditions. Minimum safety factors for the LV on 3H slope were found to be 1.8 and for the LV on 4H upstream slope were found to be 2.3 (compared with the total stress safety factor of 2.1).

## VII. Conclusions and Recommendations

39. Conclusions from the investigation are as follows:

a. Induced construction pore pressures in the left embankment ranged up to about 75 percent in a zone within backfilled excavations, El. 810+. Above this area, El. 820+ to 830+, pore pressure responses were about 50 to 60 percent. No pore pressures are present in the top 20+ feet of the fill. Pore pressures induced in the foundation overburden and shale are generally low or nonexistent.

b. Shear planes or other fractures or surfaces do not affect the integrity of the embankment. These fractures or planes apparently are a major

factor in the relatively fast response in some of the open tube devices and in the protective casings. However, all evidence, (dye tests, pump tests, the very existence of high pore pressure in the fill, and the conclusion that the fractures or planes were mechanically induced by construction equipment or in some cases by drilling and sampling procedures) suggests these planes, while numerous, are of limited extent and are not continuous.

c. The gas sampled from the fill is not coming from decaying organic material in the fill, or from the sewer in the conduit structure. The exact source of the gas is unknown but may have been present in the soil as it came from the low lying borrow deposit or may be coming from the foundation. The presence of the gas makes it very difficult to determine the magnitude of the pore pressure with open tube devices.

d. Inclinometers installed at the most critical locations indicate no significant movement of any of the slopes.

e. Stability analyses show an adequate safety factor (1.8) for the steepest existing slope and current pore pressure conditions. An adequate safety factor (2.1 to 2.3) was also found for the embankment constructed to the construction halt elevation.

f. In summary the embankment is found to be stable and is expected to perform satisfactorily as a water retention structure.

40. As a result of the investigation the following recommendations were made:

a. Given the relatively high construction pore pressures close monitoring of pore pressure and slope movement (inclinometers) should continue as the Stage II embankment is constructed. In addition because of the presence of fractures, continued close monitoring will be necessary during initial reservoir filling.

b. To facilitate ease of construction (i.e. to reduce the number of risers brought up with fill) and to resolve problems with the effect of gas on measured pore pressures, small diameter pneumatic air cells should be installed at the tips of the 3/4-inch open tube risers.

c. Stability should be reanalyzed at the time of the construction halt.

ENCLOSURE 2  
LANGSTON LABORATORIES, INC.

Laboratory Report

Date Received: December 8, 1982  
Time Received: 10:30 am  
Date Completed: December 15, 1982

Submitted by: U.S. Army Corps of Engineers  
700 Federal Building  
Kansas City, MO 64110  
Attn: Mr. Robert Dimmitt

LLI Project No.: 82-9415

Sample Description: Balloon

Sample  
Identification

Analysis

Results

Gas in Balloon


Methane

approximately 50%

Air

approximately 50%

Comments:

Approved: 

Alan Kerschen  
Laboratory Director





DEPARTMENT OF THE ARMY  
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS  
P. O. BOX 631  
VICKSBURG, MISSISSIPPI 39180  
18 January 1983

REPLY TO  
ATTENTION OF:  
Environmental Laboratory

Mr. Robert Dimmitt  
USCE, Kansas City District  
Room 816  
700 Federal Building  
601 E. 12th Street  
Kansas City, MO 64106

Dear Mr. Dimmitt:

The analysis of the gas samples taken at wells 99-9 and 99-10 are enclosed (enclosure). The analytical results shown are averages of duplicate gas chromatographic runs.

The ratio of gases in the samples is not what has normally been observed in gas mixture formed by anaerobic decomposition of organic matter. Most decomposition gases would have less methane and greater proportion of carbon dioxide. This suggests gas is not forming in the fill material itself. If there are any questions regarding the analyses please contact Richard A. Shafer at FTS 542-3943.

Sincerely,

*Philip G. Malone*  
Philip G. Malone, Ph.D.  
Geologist  
Water Supply & Waste  
Treatment Group

RESULTS OF GAS ANALYSES FROM  
PIEZOMETERS AT LONGVIEW DAM

Sample	<u>% By Volume</u>			
	Oxygen	Nitrogen	Methane	Carbon Dioxide
99-9-1	0.24	13.4	69.3	16.5
99-9-2	0.20	13.0	66.5	19.5
99-10-1	1.46	17.0	65.0	16.0
99-10-2	6.04	32.0	51.0	10.0

## APPENDIX

### Gas Sample Collection

#### Pertinant Data:

PPE 99-9

PPE 99-10

Station 99+00

Station 99+00

Range 50' US

Range 1+50' DS

Bottom of Hole El. 823'

Bottom of Hole El. 823'

Zone of sand backfill at tip 8.9'

Zone of sand backfill at tip 10'

Installation Date: 3 November 1982

Installation Date: 5 November 1982

No water encountered during drilling.

No water encountered during drilling.

No water added after backfill.

Sand backfill saturated with 4.0  
gallons of water (initial water  
level convert to El. 831.3)

A water trap device was assembled to aid in obtaining gas samples. The unit used two pressure gages, one on the line leading from the piezometer riser into the water trap and the other attached to the water trap chamber. Four valves are used to regulate flow through the device. One valve was located between the riser and the water chamber, one between the water trap chamber and the sampling tube, and two located at the bottom and top of chamber to aid in the filling and de-airing of the chamber. The chamber was filled with water to minimize the amount of air that would have to be chased out of the system to obtain a pure sample.

The procedure used to obtain gas samples is as follows:

1. If the piezometer has been winterized, remove as much diesel as possible.
2. Fill riser to the top with water.
3. Fill water trap with water removing as much trapped air as possible.
4. Attach metal connect to open tube riser using teflon tape or other sealer.
5. With all valves closed attach water trap to riser.

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ENCLOSURE 3  
DEPARTMENT OF THE ARMY  
MISSOURI RIVER DIVISION, CORPS OF ENGINEERS  
DIVISION LABORATORY  
OMAHA, NEBRASKA 68102

MRD Lab. No.

717

13 JAN 1983

Subject: Tests of Water

Project: \_\_\_\_\_

Intended Use: \_\_\_\_\_

Source of Material: Longview PD 97-3

Submitted by: \_\_\_\_\_

Date Sampled: \_\_\_\_\_

Date Received: \_\_\_\_\_

Method of Test or Specification: APHA Standard Methods

References: \_\_\_\_\_

<u>Tests Run</u>	<u>Test Result</u> ①
COD	24
PH	7.8
Suspended Solids	28
Dissolved Solids	324
Alkalinity to Phenolphthalein as $\text{CaCO}_3$	0
Alkalinity to Methyl Orange as $\text{CaCO}_3$	198
Bicarbonate ( $\text{HCO}_3$ )	242
Carbonate ( $\text{CO}_3$ )	0
Total Hardness as $\text{CaCO}_3$	153
Calcium (Ca)	30
Magnesium (Mg)	19
Carbonate Hardness as $\text{CaCO}_3$	198
Non-carbonate Hardness as $\text{CaCO}_3$	0
Sulfate ( $\text{SO}_4$ )	55
Chloride (Cl)	13
Nitrate ( $\text{NO}_3$ )	0.02
Sodium (Na)	55
Potassium (K)	3
Iron (Fe)	dissolved .024 ppm    total } 0 ppm
Manganese (Mn)	
Silica	13
Ammonia ( $\text{NH}_3$ )	0.13

## Remarks

① All values shown except pH are expressed as parts per million

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DEPARTMENT OF THE ARMY  
MISSOURI RIVER DIVISION, CORPS OF ENGINEERS  
DIVISION LABORATORY  
OMAHA, NEBRASKA 68102

MRD Lab. No.

718

13 JAN 1983

Subject: Tests of Water

Project: \_\_\_\_\_

Intended Use: \_\_\_\_\_

Source of Material: Kouquinn P299-2

Submitted by: \_\_\_\_\_

Date Sampled: \_\_\_\_\_

Date Received: \_\_\_\_\_

Method of Test or Specification: APHA Standard Methods

References: \_\_\_\_\_

<u>Tests Run</u>		<u>Test Result</u> ①
COD		79
PH		7.3
Suspended Solids		275
Dissolved Solids		1170
Alkalinity to Phenolphthalein as $\text{CaCO}_3$		0
Alkalinity to Methyl Orange as $\text{CaCO}_3$		785
Bicarbonate ( $\text{HCO}_3$ )		956
Carbonate ( $\text{CO}_3$ )		0
Total Hardness as $\text{CaCO}_3$		1025
Calcium (Ca)		314
Magnesium (Mg)		58
Carbonate Hardness as $\text{CaCO}_3$		785
Non-carbonate Hardness as $\text{CaCO}_3$		240
Sulfate ( $\text{SO}_4$ )		3
Chloride (Cl)		18
Nitrate ( $\text{NO}_3$ )		0.00
Sodium (Na)		24
Potassium (K)		7
Iron (Fe)	} dissolved 0.146 ppm 0.649 ppm	} 43.02 10.6
Manganese (Mn)		
Silica		18
Ammonia ( $\text{NH}_3$ )		1.06

## Remarks

- ① All values shown except PH are expressed as parts per million.  
② See back

83/27

DEPARTMENT OF THE ARMY  
MISSOURI RIVER DIVISION, CORPS OF ENGINEERS  
DIVISION LABORATORY  
OMAHA, NEBRASKA 68102

MRD Lab. No. 719

13 JAN 1983

Subject: Tests of Water

Project: \_\_\_\_\_

Intended Use: \_\_\_\_\_

Source of Material: Konguian P2 99-4

Submitted by: \_\_\_\_\_

Date Sampled: \_\_\_\_\_

Date Received: \_\_\_\_\_

Method of Test or Specification: APHA Standard Methods

References: \_\_\_\_\_

Tests Run	Test Result <sup>①</sup>
COD	77
PH	8.1
Suspended Solids	93
Dissolved Solids	680
Alkalinity to Phenolphthalein as CaCO <sub>3</sub>	0
Alkalinity to Methyl Orange as CaCO <sub>3</sub>	486
Bicarbonate (HCO <sub>3</sub> )	593
Carbonate (CO <sub>3</sub> )	0
Total Hardness as CaCO <sub>3</sub>	286
Calcium (Ca)	83
Magnesium (Mg)	19
Carbonate Hardness as CaCO <sub>3</sub>	486
Non-carbonate Hardness as CaCO <sub>3</sub>	0
Sulfate (SO <sub>4</sub> )	53
Chloride (Cl)	26
Nitrate (NO <sub>3</sub> )	0.00
Sodium (Na)	107
Potassium (K)	16
Iron (Fe)	} dissolved 0.171 ppm 1233 ppm } total 3
Manganese (Mn)	
Silica	22
Ammonia (NH <sub>3</sub> )	0.00

## Remarks

① All values shown except pH are expressed as parts per million

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DEPARTMENT OF THE ARMY  
MISSOURI RIVER DIVISION, CORPS OF ENGINEERS  
DIVISION LABORATORY  
OMAHA, NEBRASKA 68102

MRO Lab. No.

720

13 JAN 1983

Subject: Tests of Water

Project: \_\_\_\_\_

Intended Use: \_\_\_\_\_

Source of Material: Longview P2-99-3

Submitted by: \_\_\_\_\_

Date Sampled: \_\_\_\_\_

Date Received: \_\_\_\_\_

Method of Test or Specification: APHA Standard Methods

References: \_\_\_\_\_

<u>Tests Run</u>	<u>Test Result</u> ①
COD	63
PH	7.3
Suspended Solids	155
Dissolved Solids	632
Alkalinity to Phenolphthalein as $\text{CaCO}_3$	0
Alkalinity to Methyl Orange as $\text{CaCO}_3$	550
Bicarbonate ( $\text{HCO}_3$ )	671
Carbonate ( $\text{CO}_3$ )	0
Total Hardness as $\text{CaCO}_3$	516
Calcium (Ca)	158
Magnesium (Mg)	30
Carbonate Hardness as $\text{CaCO}_3$	550
Non-carbonate Hardness as $\text{CaCO}_3$	0
Sulfate ( $\text{SO}_4$ )	26
Chloride (Cl)	16
Nitrate ( $\text{NO}_3$ )	0.00
Sodium (Na)	19
Potassium (K)	4
Iron (Fe)	dissolved 0.073 ppm total } 57.36
Manganese (Mn)	
Silica	14
Ammonia ( $\text{NH}_3$ )	0.20

## Remarks

① All values shown except pH are expressed as parts per million

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DEPARTMENT OF THE ARMY  
MISSOURI RIVER DIVISION, CORPS OF ENGINEERS  
DIVISION LABORATORY  
OMAHA, NEBRASKA 68102

HRD Lab. No. 721  
13 JAN 1983

Subject: Tests of Water

Project: \_\_\_\_\_

Intended Use: \_\_\_\_\_

Source of Material: Longview P2-99-7

Submitted by: \_\_\_\_\_

Date Sampled: \_\_\_\_\_

Date Received: \_\_\_\_\_

Method of Test or Specification: APHA Standard Methods

References: \_\_\_\_\_

<u>Tests Run</u>	<u>Test Result</u> ①
COD	68
PH	7.8
Suspended Solids	33
Dissolved Solids	1010
Alkalinity to Phenolphthalein as $\text{CaCO}_3$	0
Alkalinity to Methyl Orange as $\text{CaCO}_3$	872
Bicarbonate ( $\text{HCO}_3$ )	1064
Carbonate ( $\text{CO}_3$ )	0
Total Hardness as $\text{CaCO}_3$	477
Calcium (Ca)	137
Magnesium (Mg)	33
Carbonate Hardness as $\text{CaCO}_3$	872
Non-carbonate Hardness as $\text{CaCO}_3$	0
Sulfate ( $\text{SO}_4$ )	75
Chloride (Cl)	50
Nitrate ( $\text{NO}_3$ )	0.00
Sodium (Na)	200
Potassium (K)	11
Iron (Fe)	} dissolved 0.805 ppm total } 0.8
Manganese (Mn)	
Silica	2.8
Ammonia ( $\text{NH}_3$ )	0.96

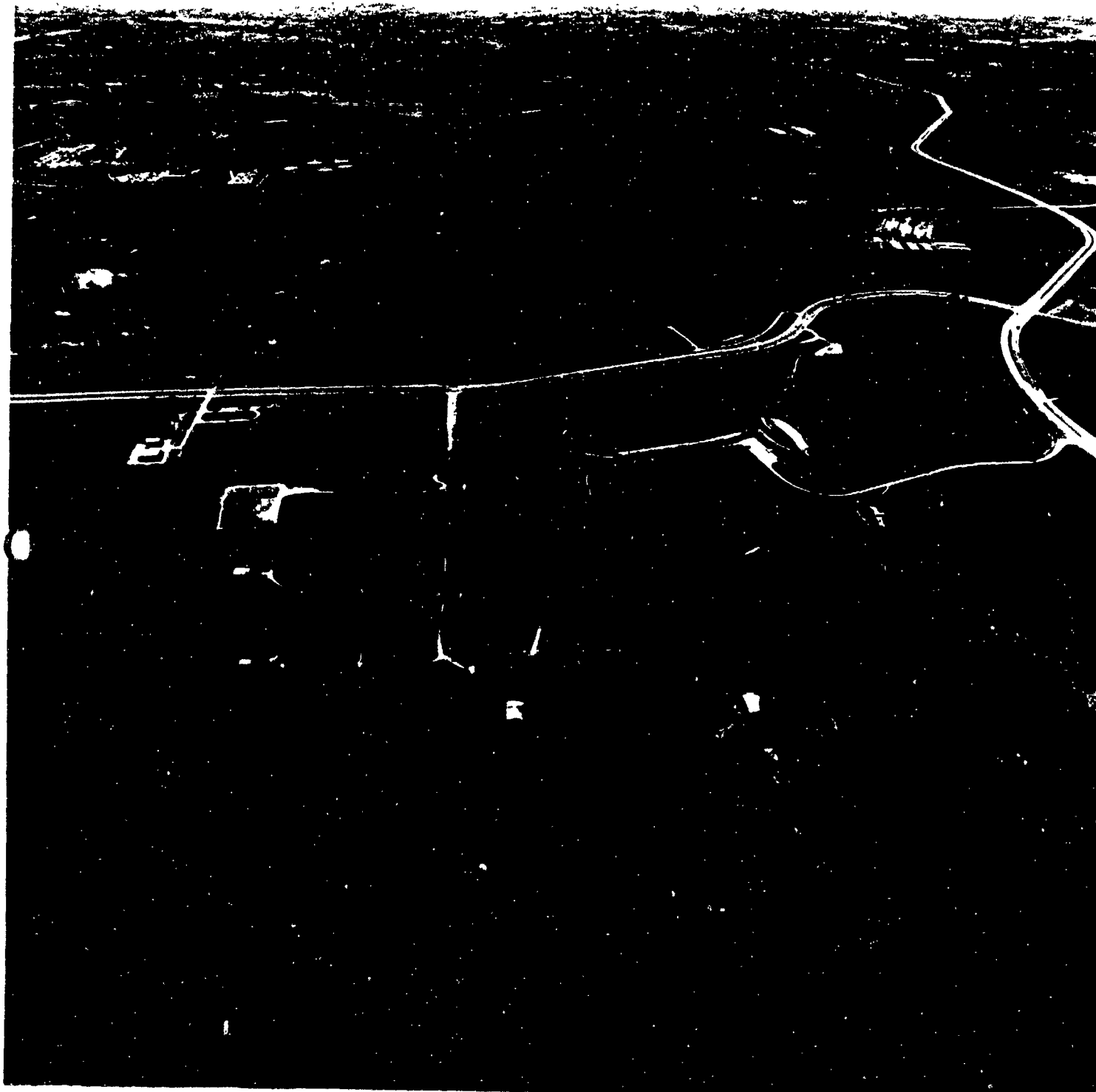
Remarks

① All values shown except PH are expressed as parts per million  
725 Small



**PHOTOGRAPHS**

**PHOTOGRAPHS**

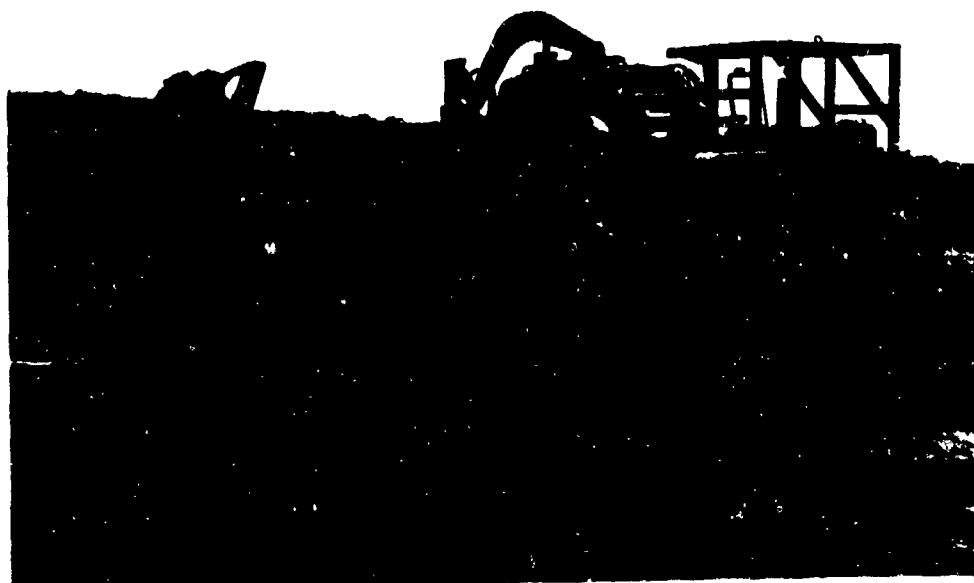


1. March 1990. Overview of completed embankment from downstream of right abutment.

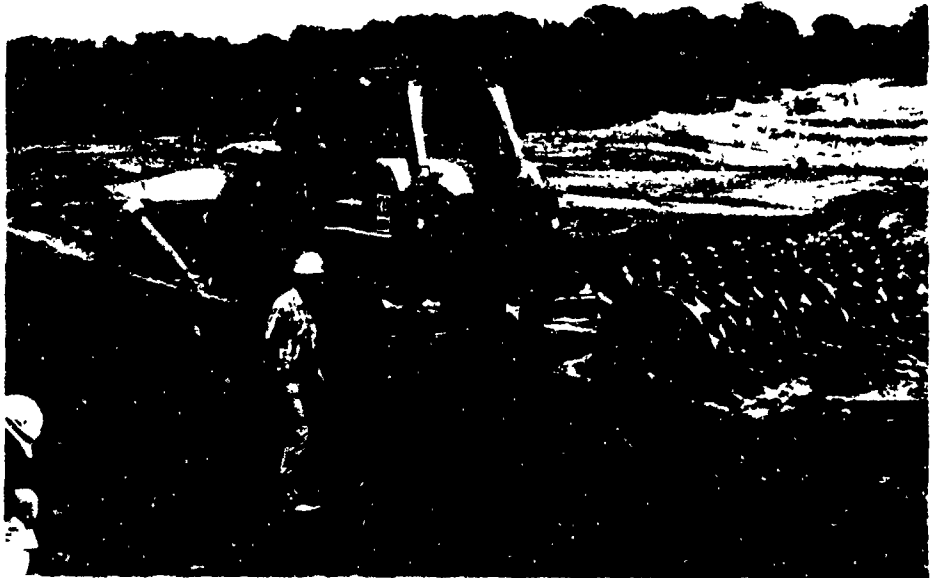
CONSTRUCTION EQUIPMENT



2. 1983. Two Caterpillar D-9s loading Cat 631 scrapers in upstream valley borrow area.



3. 09 October 1981. Cat 5310 scraper.



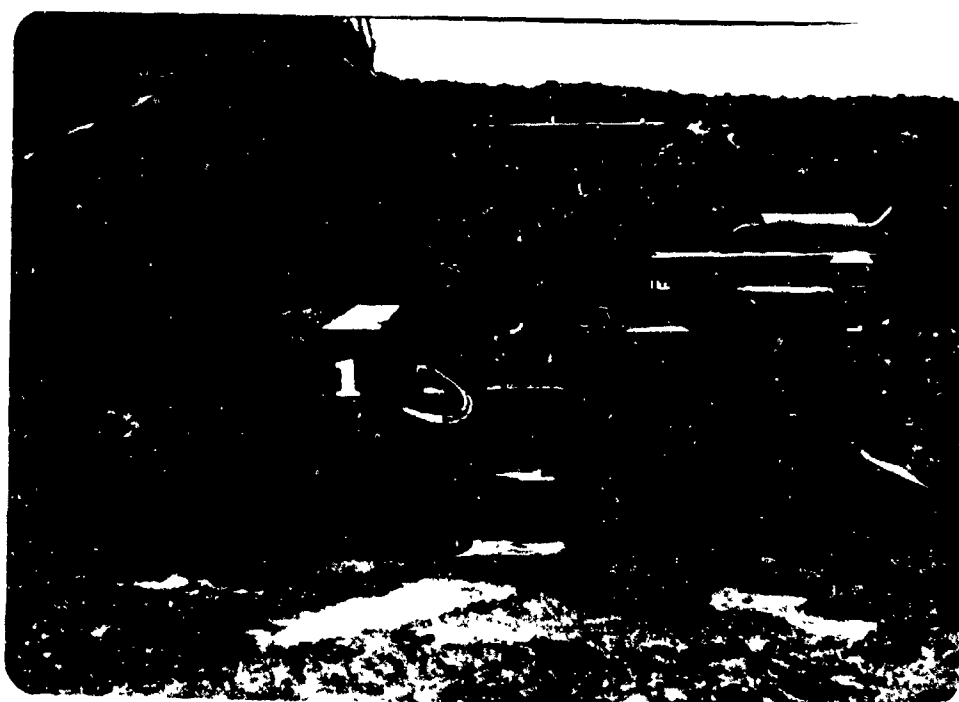
4. 26 July 1983. Cat D-9H compacting haul road ramp with tandem sheepsfoot roller Looking toward downstream right abutment.



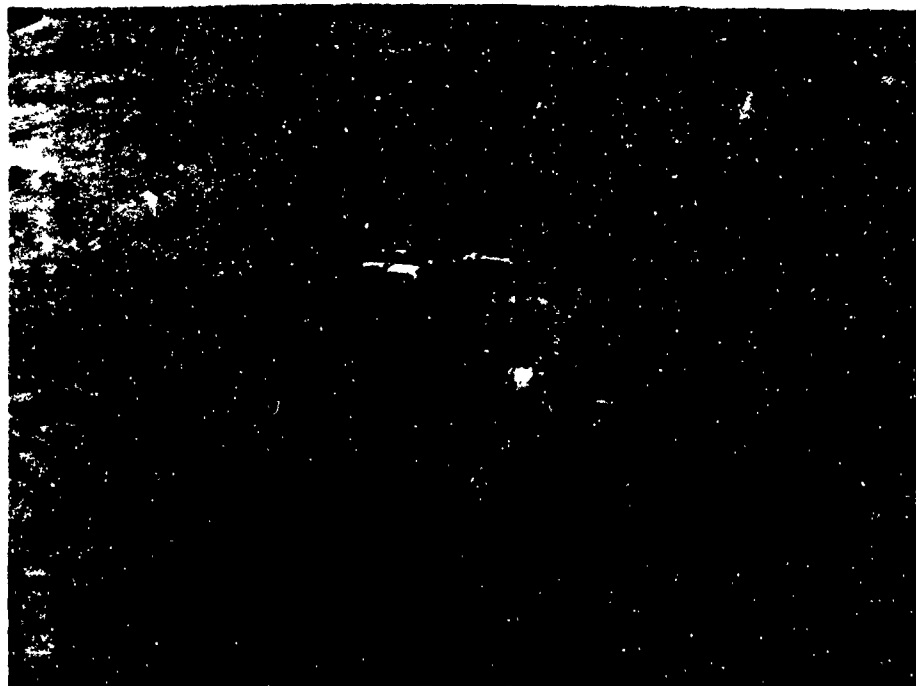
5. 10 July 1983. Compacting impervious fill around sump in cutoff trench with a "BARCO" gas powered compactor.



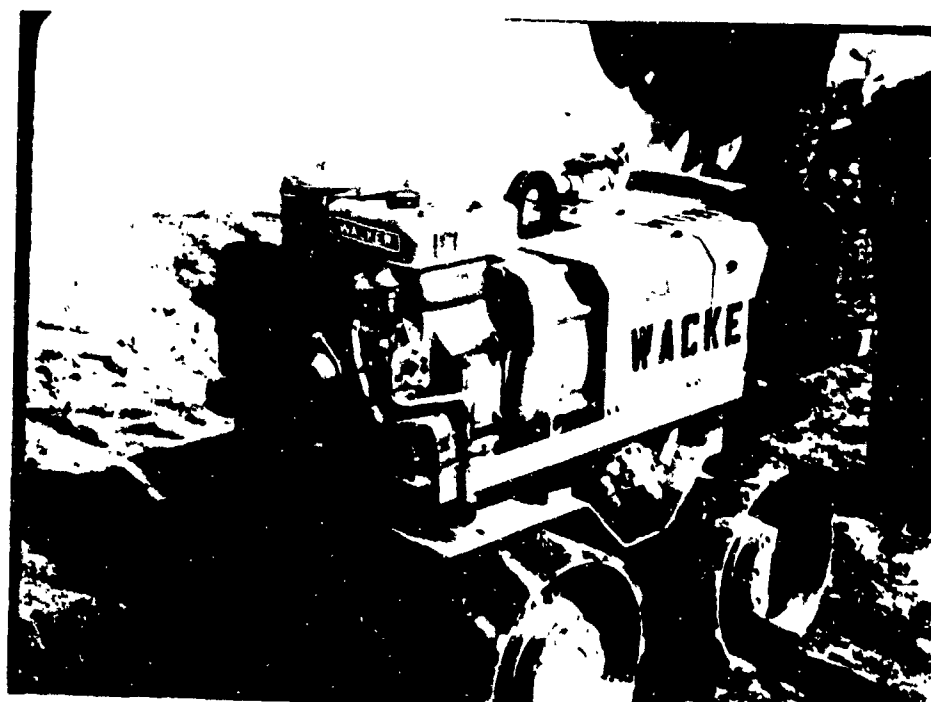
6. 06 July 1983. Spreader box mounted on a Cat D3 used for placing the 6 foot wide pervious drain.



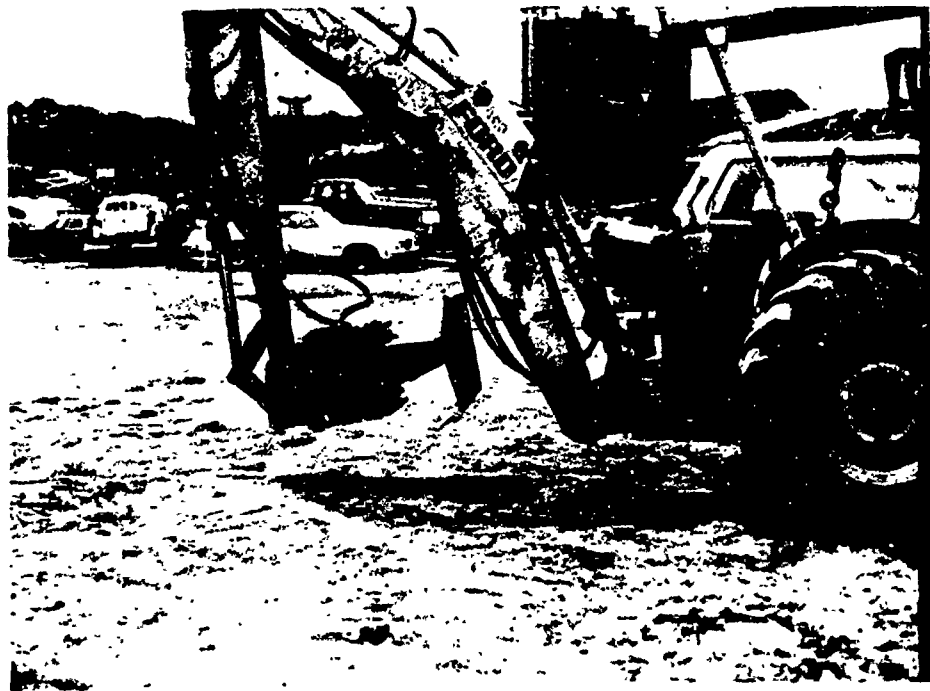
7. June 1980. Raygo "Rascal" 410-A 5 ton vibratory roller used to compact pervious fill. , Looking downstream in cutoff trench.



8. 07 June 1983. "Wacker" DVU4001 small vibratory plate compactor used to compact pervious fill in restricted areas.



9. 07 May 1981. Small 4 wheel "Wacker" vibratory roller. Attempts were made to use this roller to compact pervious around conduit. Roller buried itself frequently and could not be used.



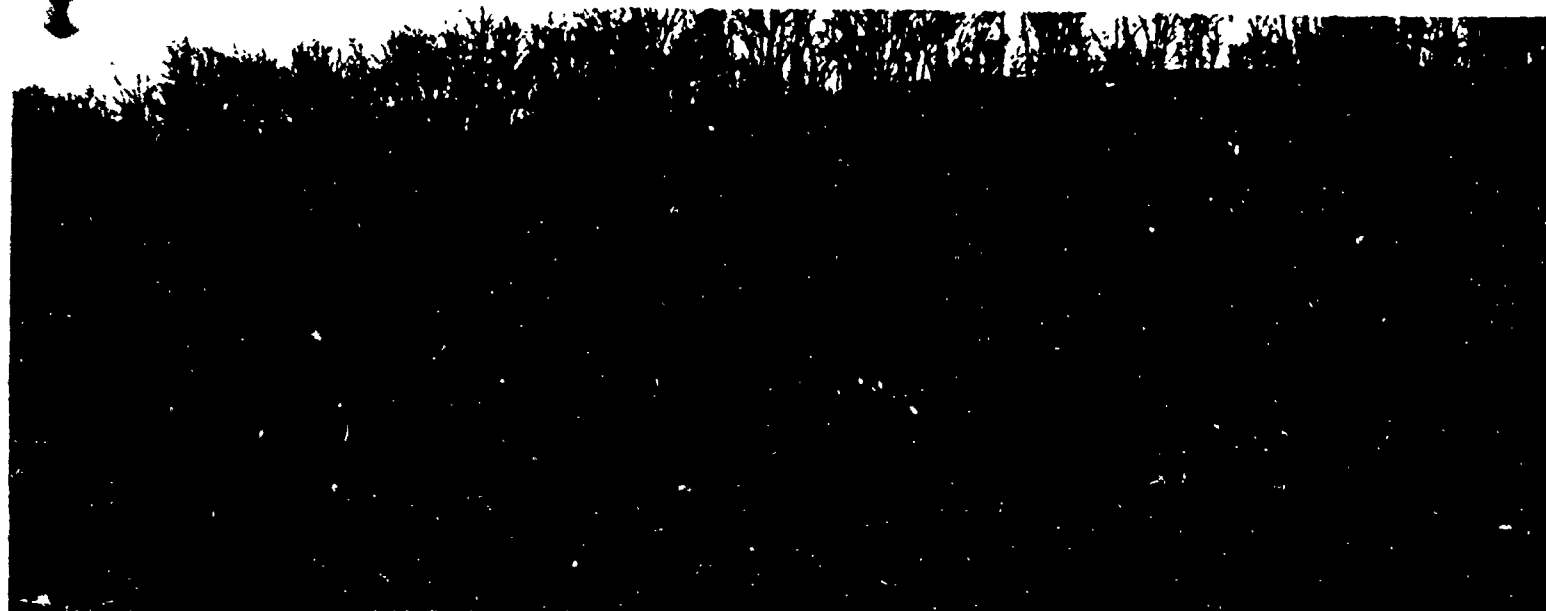
10. 11 June 1981. Backhoe mounted vibratory plate used to compact pervious fill along the right side of the conduit.



11. 18 September 1981. Wheel rolling upstream cutoff trench contact with left abutment Sni-A-Bar Limestone with Cat 920 front end loader.

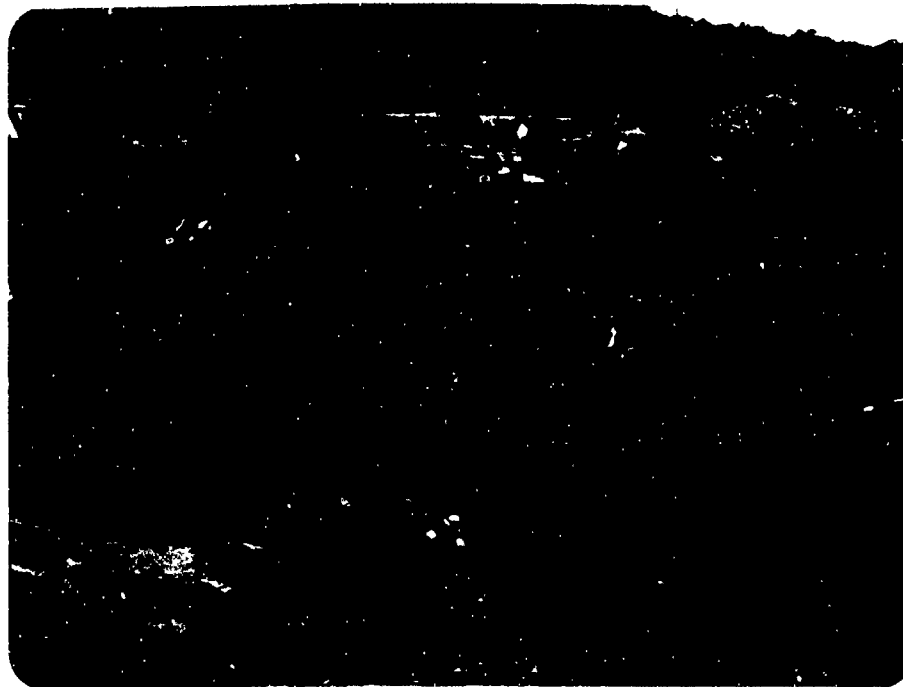


12. 18 July 1983. Record control sample being taken.  
STAGE I EMBANKMENT CONSTRUCTION



13. 05 March 1980. Looking west at slide on left side of  
outlet works excavation. Slide occurred on 18 November 1979  
between approximate outlet works stations 55+00 and 57+50.  
Estimated volume approximately 39000 cubic yards.

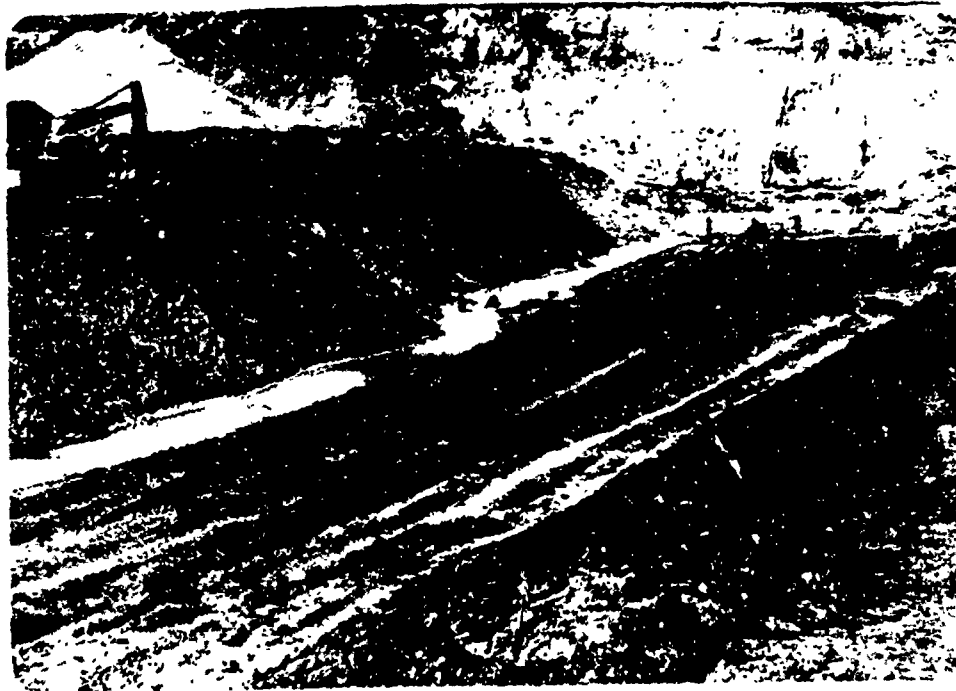




14. 29 May 1980. Curtain grouting in cutoff trench.  
Looking toward downstream left abutment.



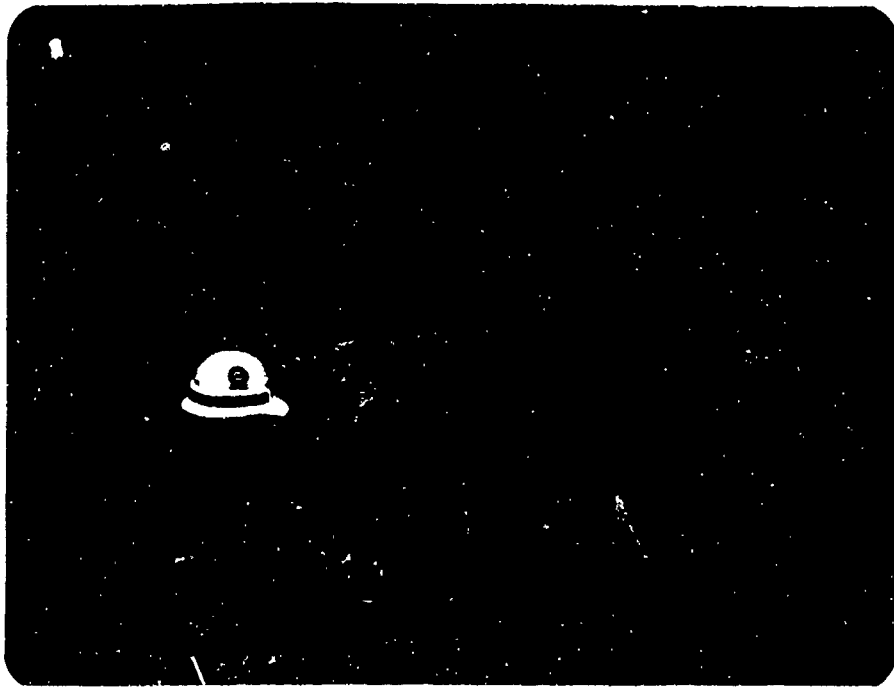
15. July 1980. Removal of unacceptably wet impervious fill  
material and excavation for placement of pervious on the  
downstream side of the cutoff trench. Looking toward left  
abutment from right abutment.



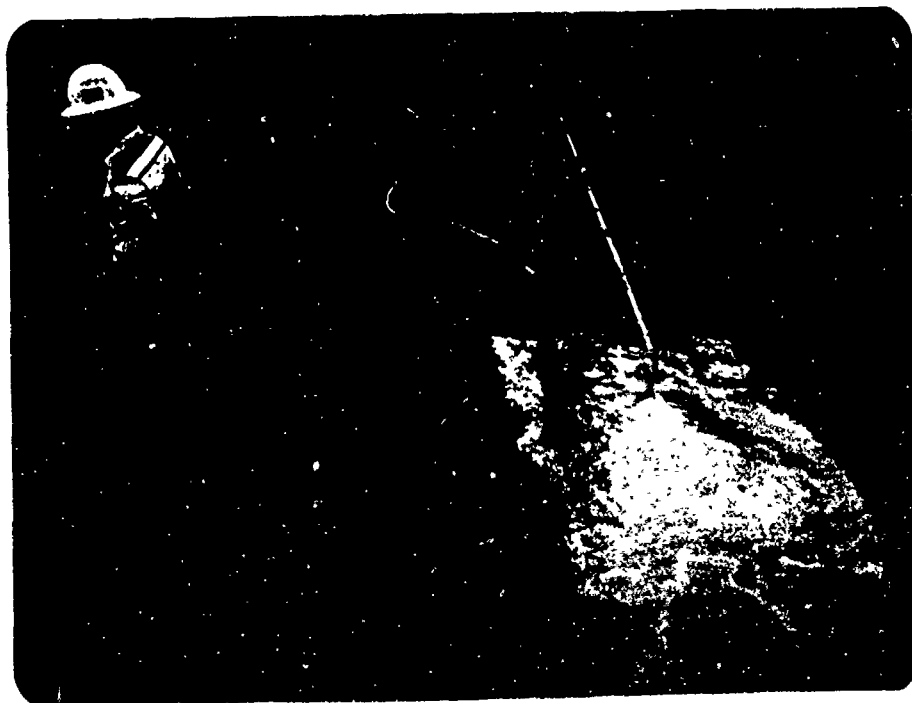
16. July 1980. Impervious and pervious fill placed in cutoff trench against right abutment. Looking toward right abutment.



17. 21 July 1980. Outlet works ogee section and 45 degree bend of twin sewer conduits. Looking downstream.



18. 14 August 1980. Stair stepped surface of heavily fractured Middle Creek limestone of right abutment after excavation and cleaning. Looking downstream.



19. 14 August 1980. Placement of dental concrete and slush grout to seal the surface of Middle Creek limestone of the right abutment. Looking downstream.



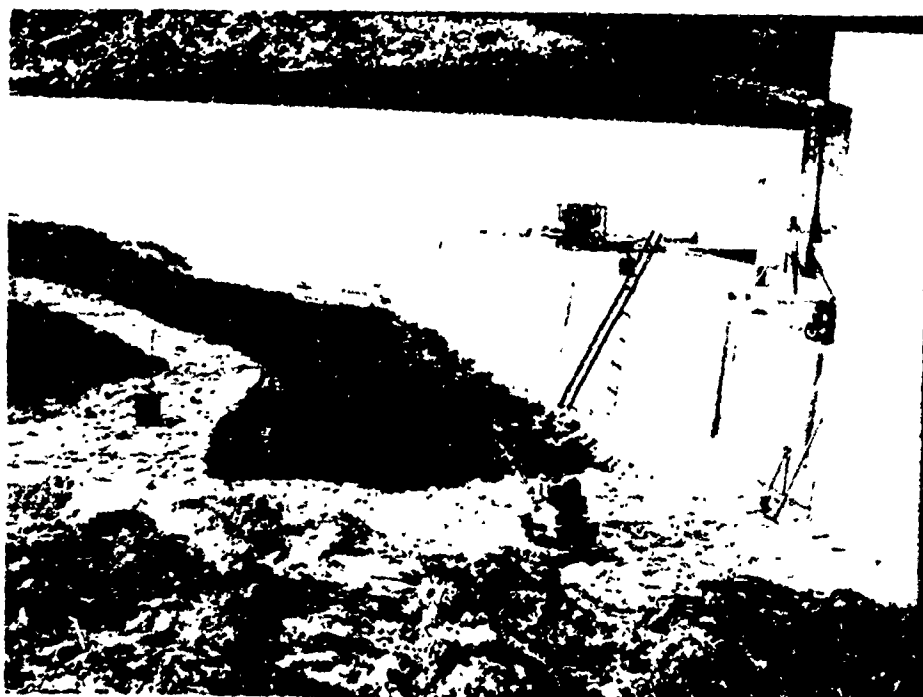
20. 21 October 1980. Overview of fill placement, conduit construction, and left abutment cutoff trench from right abutment.



21. 06 May 1981. Selective loading with rock rake of left abutment Sni-A-Bar limestone to make rockfill.



22. 21 May 1981. Overview of conduit backfill, fill placement, and right abutment from left abutment.



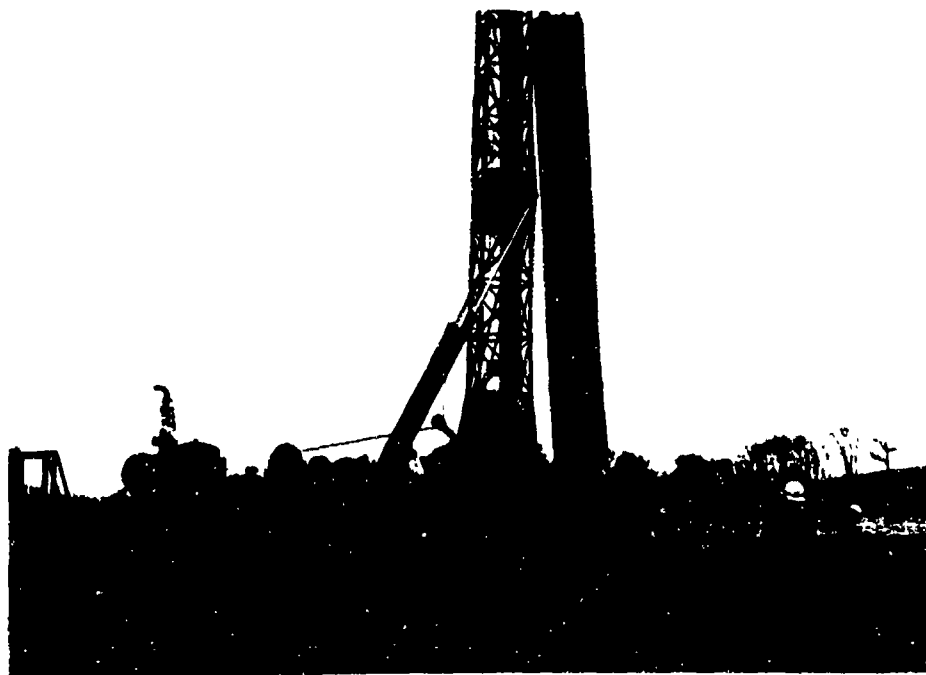
23. 21 July 1981. First lift of impervious fill being placed over pervious on upstream side of bend in twin sewer conduits at approximate outlet works station 54+00. Looking toward left abutment.



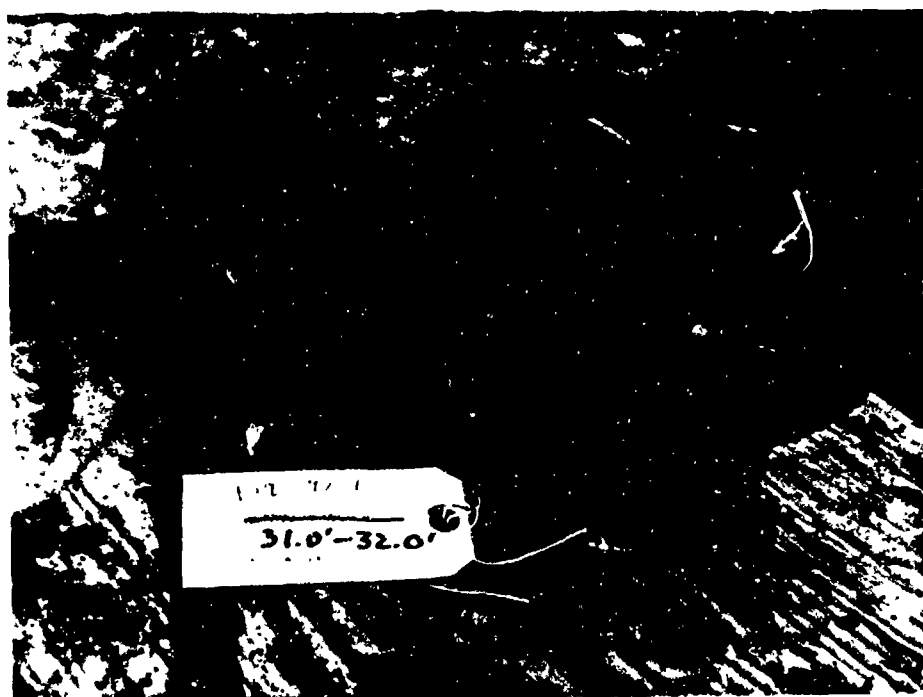
24. 18 September 1982. Left abutment and cutoff trench.



25. 21 October 1982. Downstream impervious stockpile.  
Looking downstream of right abutment.



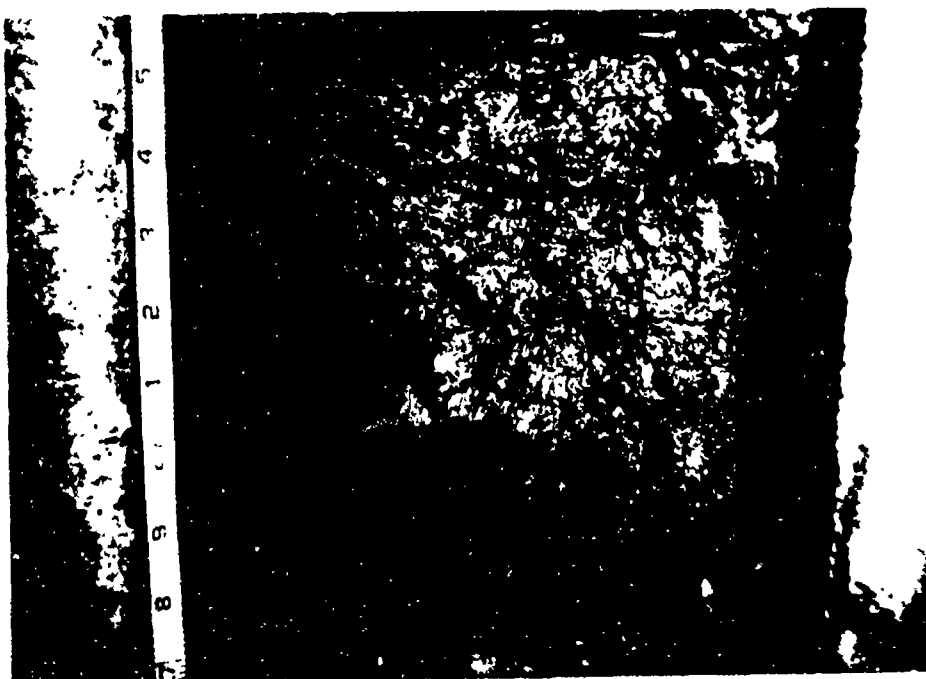
26. 16 November 1982. Stage I embankment investigation. Lowering inspection casing into 30 inch diameter observation hole (OH). Looking toward downstream right abutment.



27. November 1982. Sample showing typical smooth surface texture, probably caused by pumping action of loaded scraper; PPE-97-15, Depth 31-32 feet; Elevation 837.7 to 838.7.

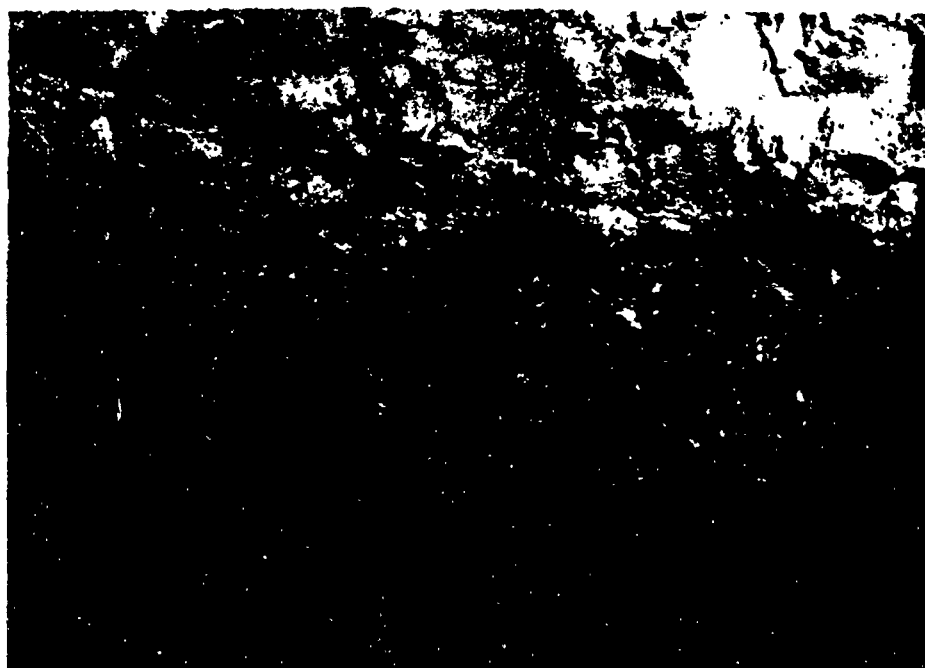


28. November 1982. Sample showing typical raindrop-type texture, probably mechanically induced from construction equipment; PPE-97-15; Depth 29.0 to 30.8 feet; Elevation 838.9 to 840.7.

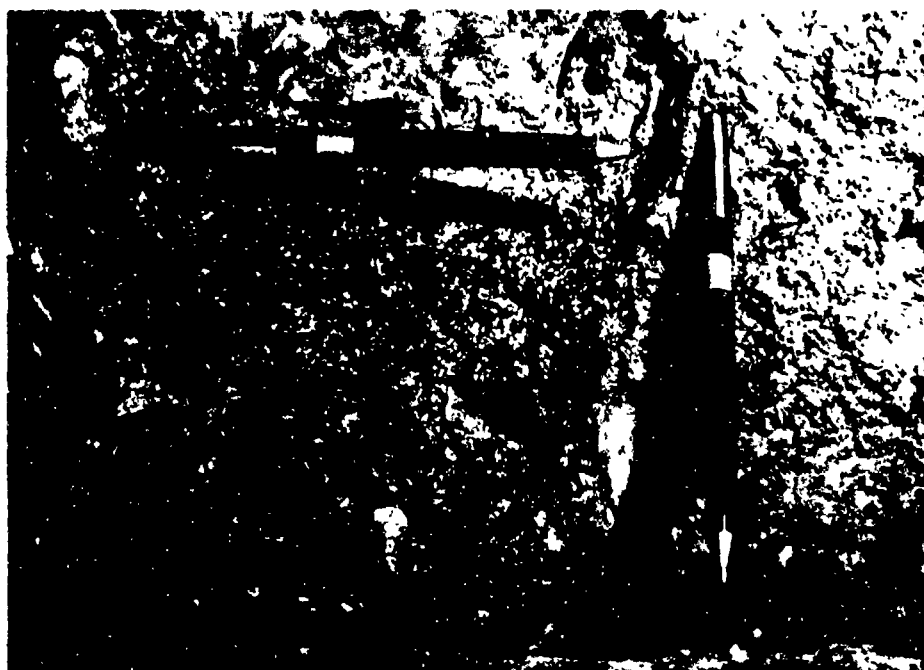


29. November 1982. First wet area encountered in OH-1; Depth 23 feet; Elevation 847.

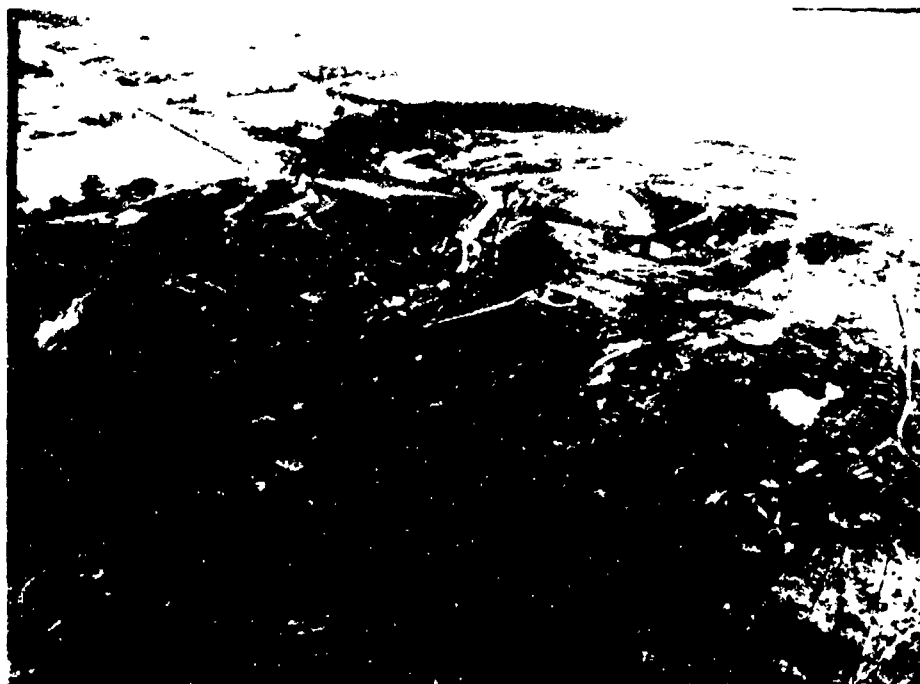




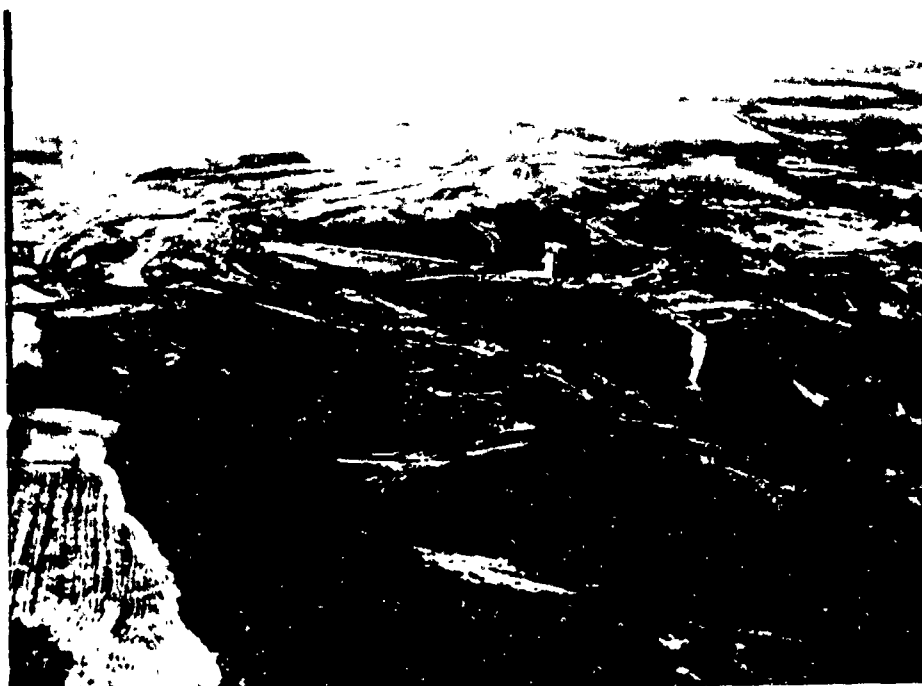
30. November 1982. One foot above area in previous photo; exposed boundary layer with the upper material moist and seepage exiting from lower material; OH-1; Depth 22 feet; Elevation 848.



31. November 1982. Wet area encountered in OH-1; Depth 34 feet; Elevation 836.



32. 24 November 1982. End of Stage I construction. Aerial view of construction from downstream of left abutment looking southeast.

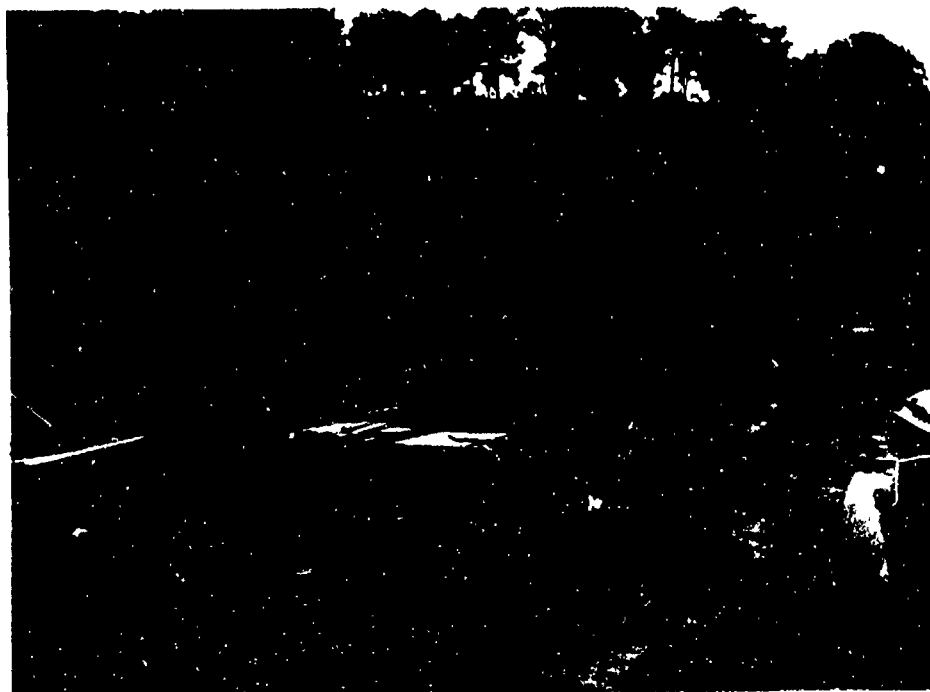


33. 24 November 1982. End of Stage I construction. Aerial view of construction from downstream of right abutment looking southwest.

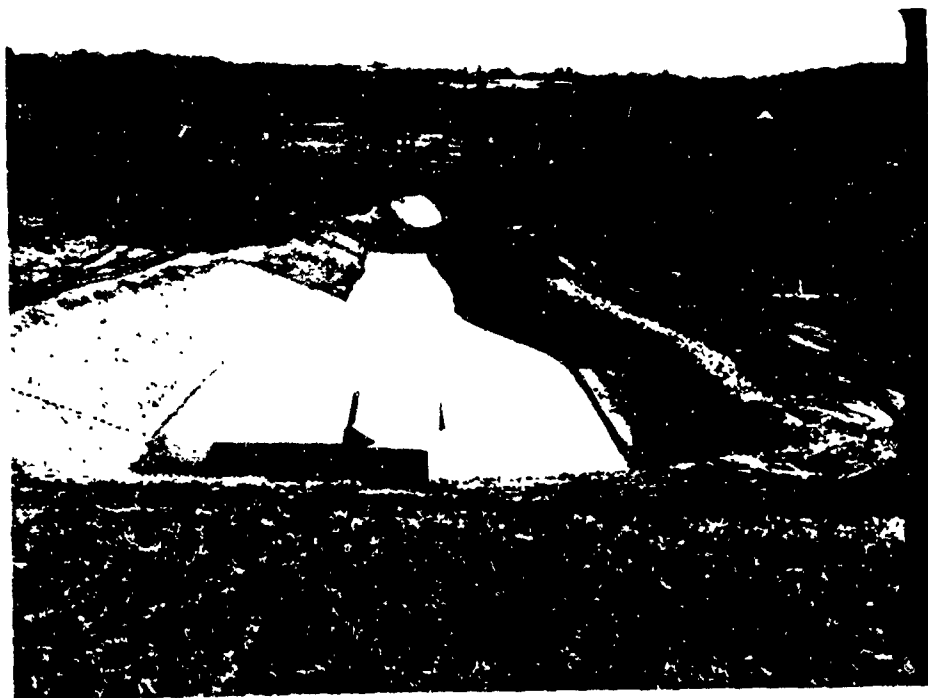
STAGE II EMBANKMENT CONSTRUCTION



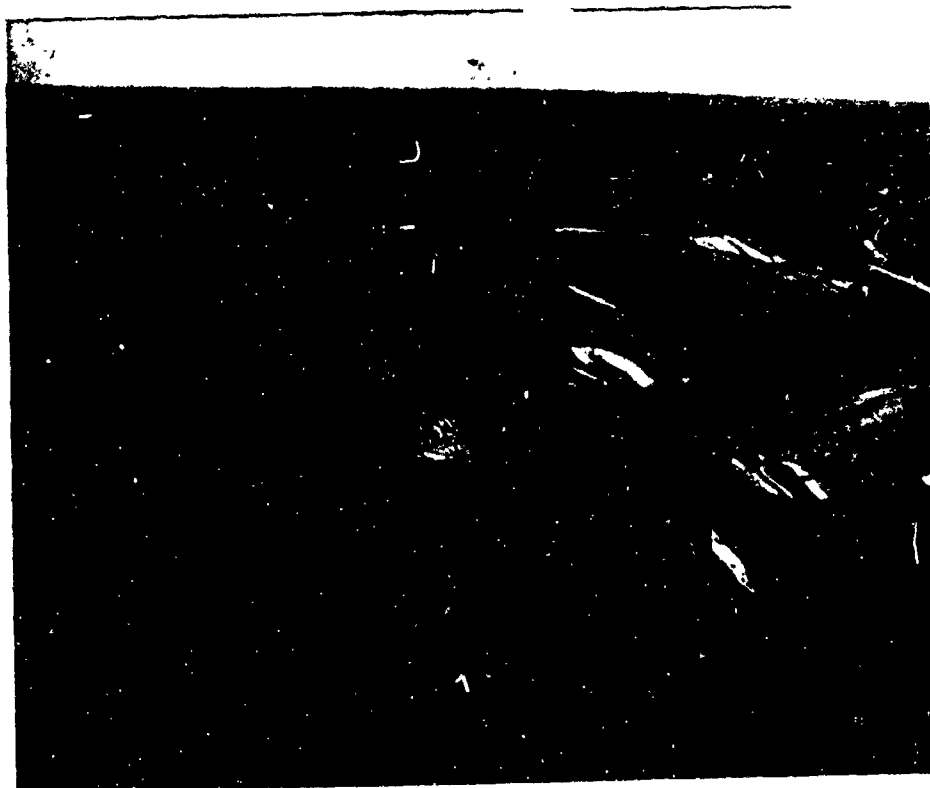
34. January 1983. Ignition of gas from PPE99-9.



35. 10 June 1983. Impervious planket against upstream right abutment.



36. 17 June 1983. Completed outlet works and stilling basin. Looking downstream.



37. 06 July 1983. Aerial view of embankment, cofferdam, and closure section construction from downstream of right abutment.



38. 06 July 1983. Aerial view of embankment and closure section construction from upstream left abutment.



39. 09 July 1983. Cleanup of cutoff trench and impervious placement in closure section.



40. 12 July 1983. Pervious placed at downstream edge of cutoff trench in closure section. Looking toward right embankment.



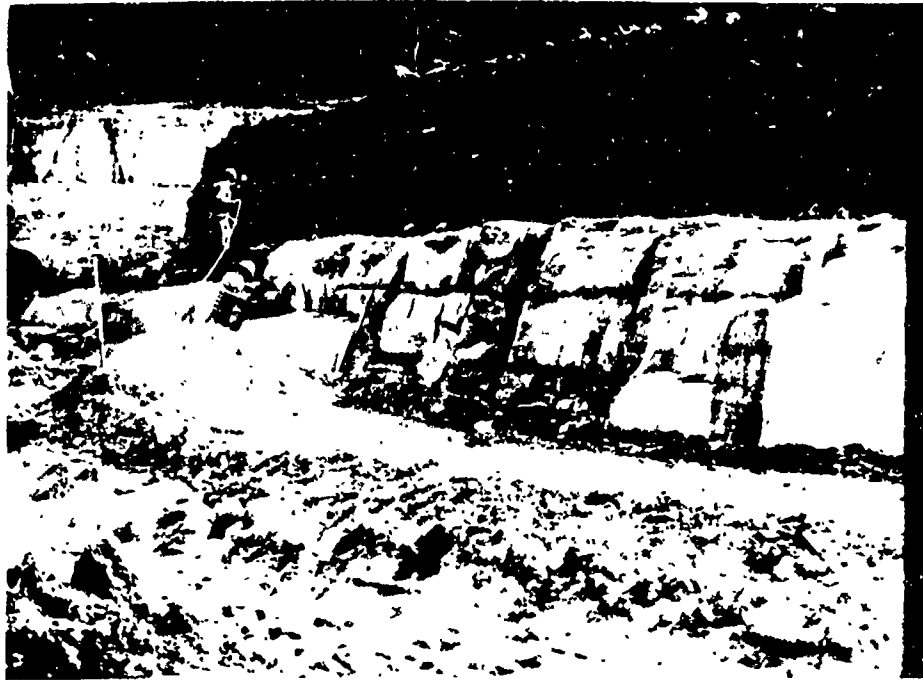
41. 18 July 1983. View of closure section showing pervious placement at downstream side of, cutoff trench and in old river and diversion channels. Looking downstream.



42. 22 July 1983. Pervious blanket placed over old river channel in closure section. Looking downstream.



43. September 1983. Closure section embankment construction. Looking toward downstream right abutment from left abutment.



44. 13 September 1983. Downstream right abutment.



45. 13 September 1983. Upstream right abutment.

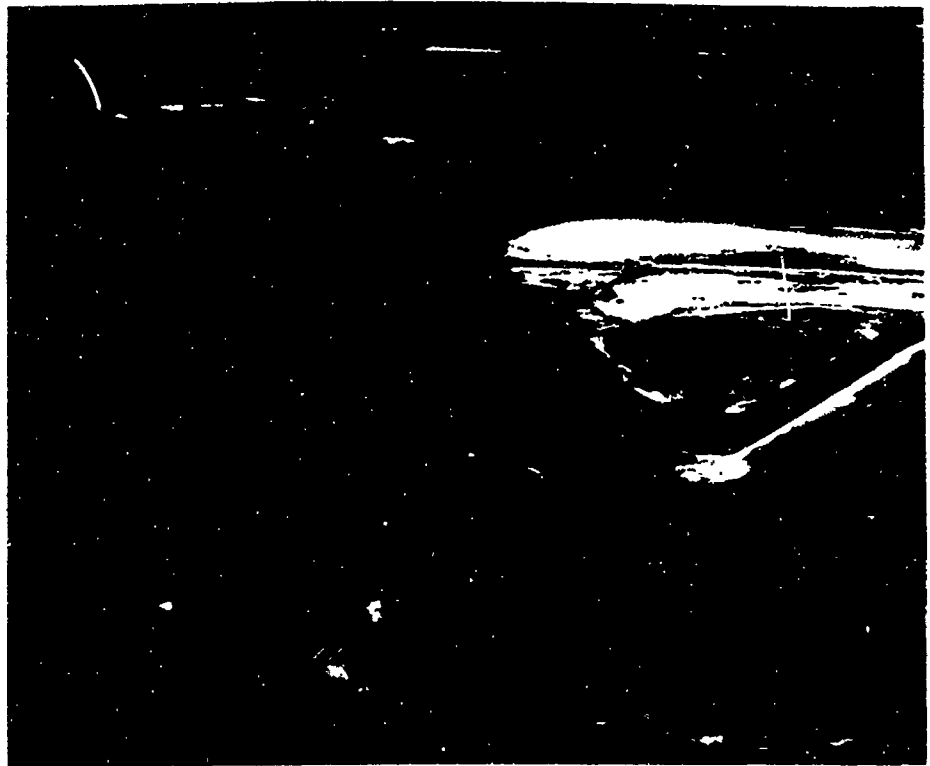




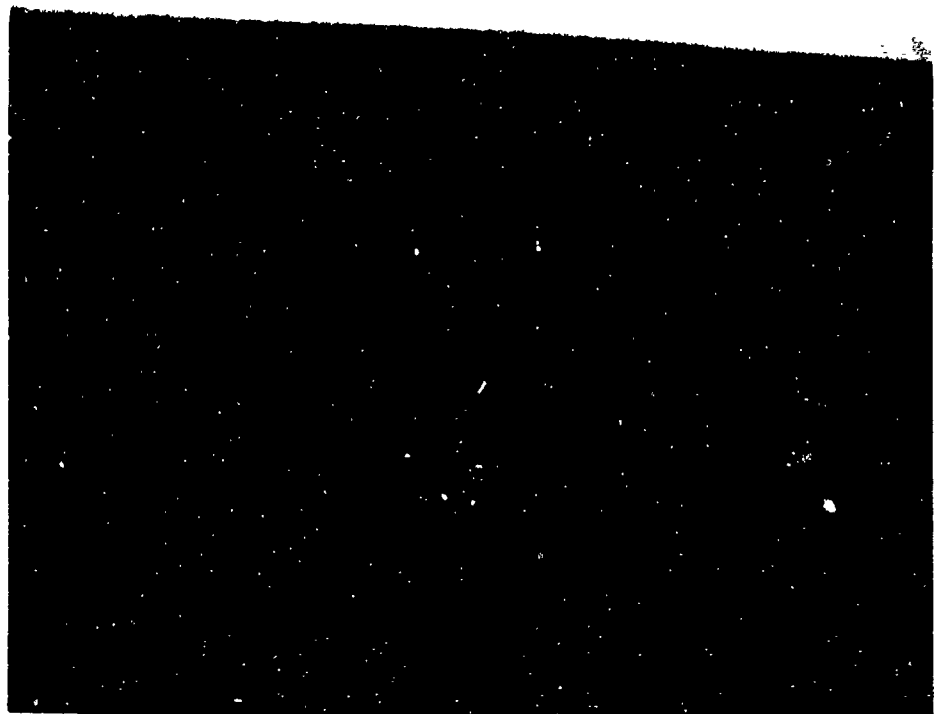
46. 26 October 1983. Fill placement in closure section.  
Looking toward right abutment.



47. 24 May 1984. Rockfill in clay blanket area on upstream  
right abutment. Looking upstream.



48. May 1984. Aerial view of embankment after completion of closure section fill placement. From above left abutment looking toward right abutment.



49. May 1984. Aerial view of embankment looking upstream from downstream of the right abutment. The east borrow is visible in the center of the photo and the west borrow is at the upper right center.



50. May 1984. Aerial view of borrow areas from south of the west borrow area looking north toward the east borrow area and the embankment.



51. 16 July 1984. Upstream and downstream random zone fill placement. Looking toward right abutment.



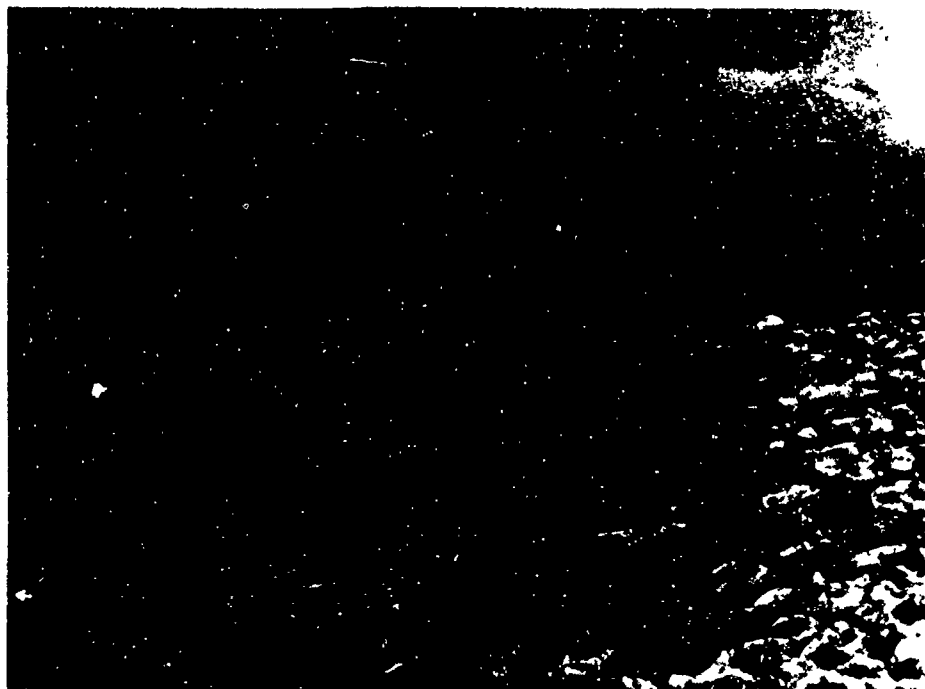
52. 18 September 1984. Upstream slope before final grading. Looking toward upstream left abutment.



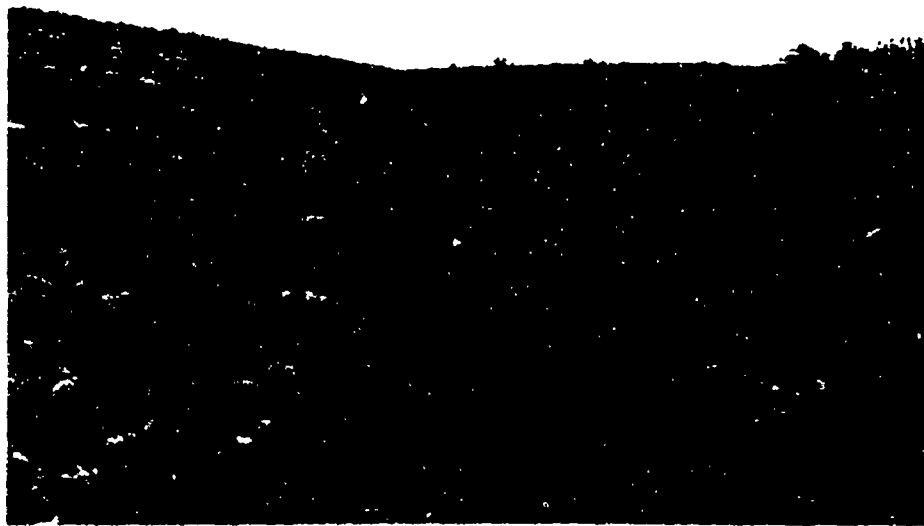
53. 26 November 1984. Downstream left abutment grouted gutter construction. Looking upstream.



54. 25 March 1985. Front end loaders placing riprap on upstream face of embankment. Looking downstream.



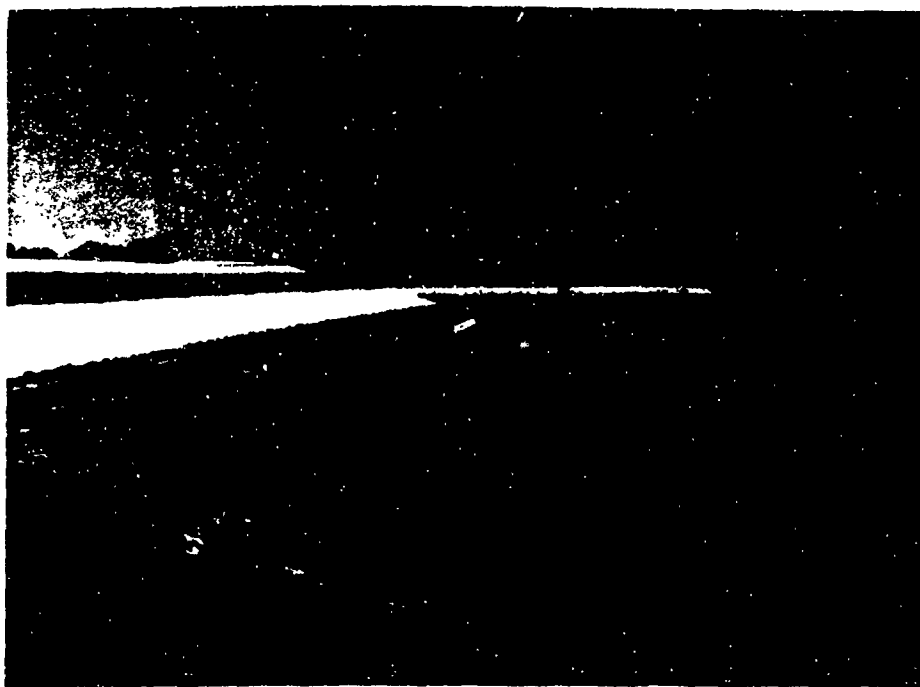
55. 26 March 1985. Dressing of riprap adjacent to bridge abutment using large backhoe.



56. 26 March 1985. Completed upstream slope protection.  
Looking toward upstream right abutment.



57. 23 May 1985. Completed slope protection at crest of  
embankment. Looking toward left abutment.



58. September 1985. Completed upstream slope from tower bridge. Looking toward upstream right abutment.



59. September 1985. Completed, downstream right abutment from stilling basin parking lot.



60. 24 September 1985. Eight foot flat bottom toe ditch. Looking toward stilling basin.

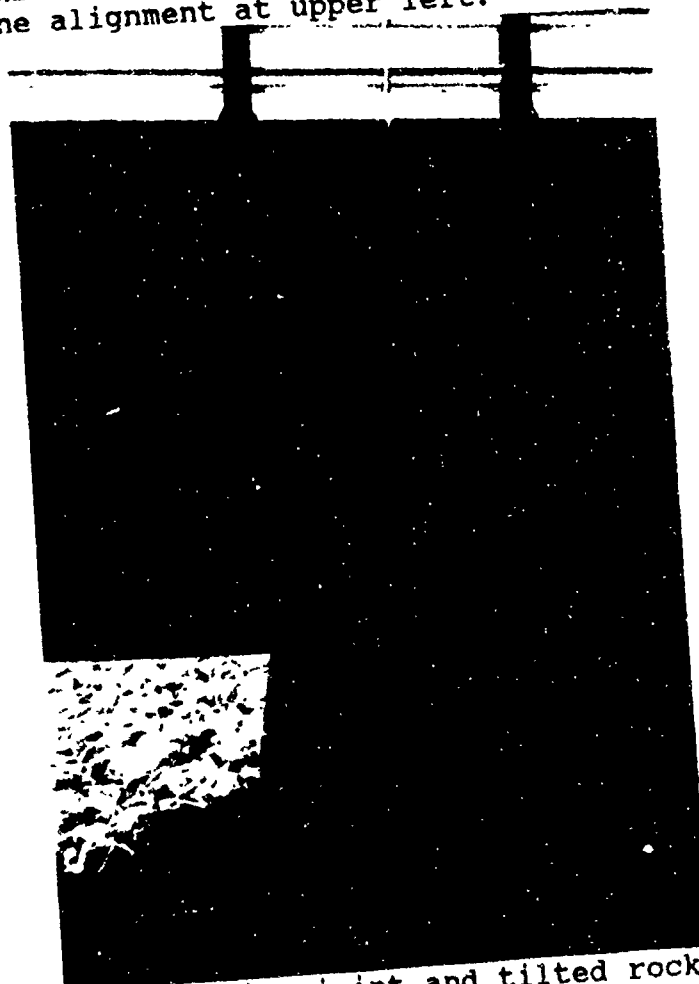


61. 24 September 1985. "Enka-mat" erosion control fabric lined ditch above grouted gutter on downstream left abutment.





62. 05 March 1986. Downstream right abutment contact showing cleared sewer line alignment at upper left.



63. 22 April 1986. Closed expansion joint and tilted rocker due to movement of tower bridge abutment.



64. April 1986. Crown crack and leakage along left wall at monolith joint between tower and conduit.



65. 19 May 1986. Installation of finger drains into pervious blanket at embankment toe.



66. 19 May 1986. Completed finger drain installation at embankment toe..



67. 10 July 1986. Completed french drain installed in upstream right abutment clay blanket to intercept seepage from limestone.



68. 18 July 1986. Aerial view of completed embankment. Looking upstream from downstream of the right abutment.



69. 19 September 1986. Upstream slope protection with pool at multipurpose elevation 891. , Looking toward upstream left abutment from right abutment.



70. 04 October 1986. Upstream slope with pool at elevation 897. Looking toward right abutment from left abutment.



71. 07 April 1987. Left abutment slope protection with the pool at multipurpose elevation 891.



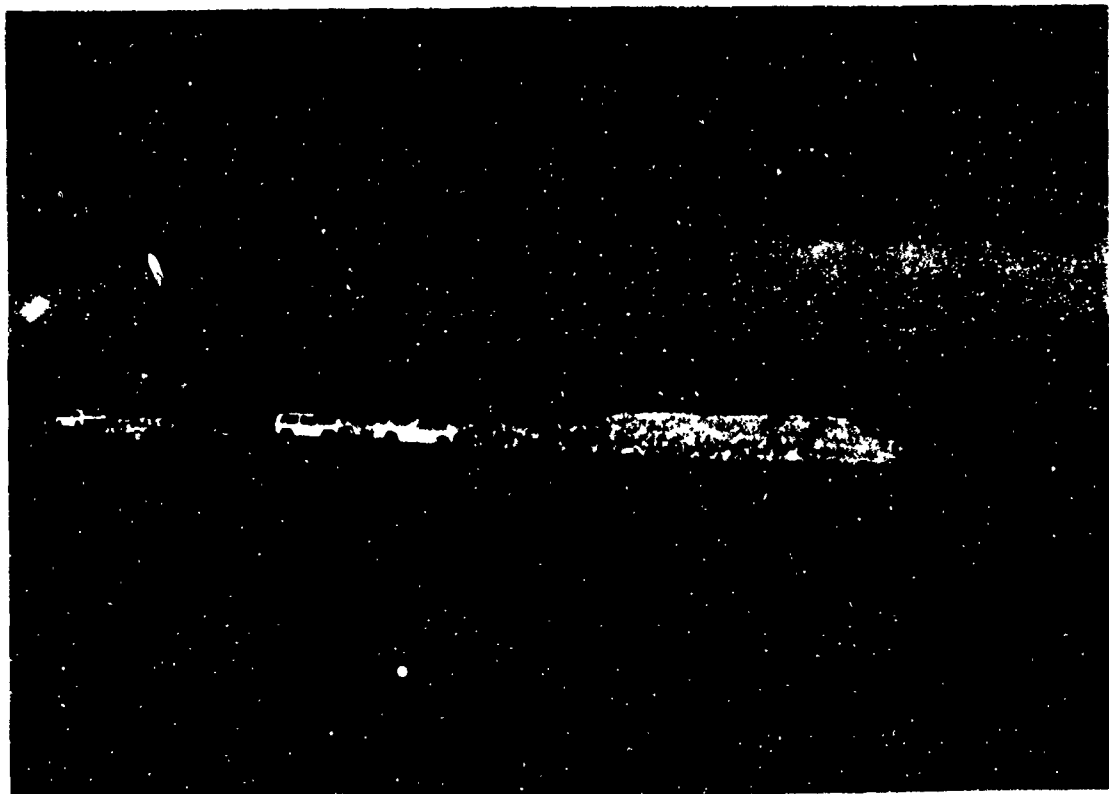
72. 07 April 1987. Completed upstream right abutment clay blanket area.



73. 19 September 1986. Downstream left abutment contact and stilling basin.



74. 07 April 1987. Completed downstream slope. Looking toward downstream left abutment.

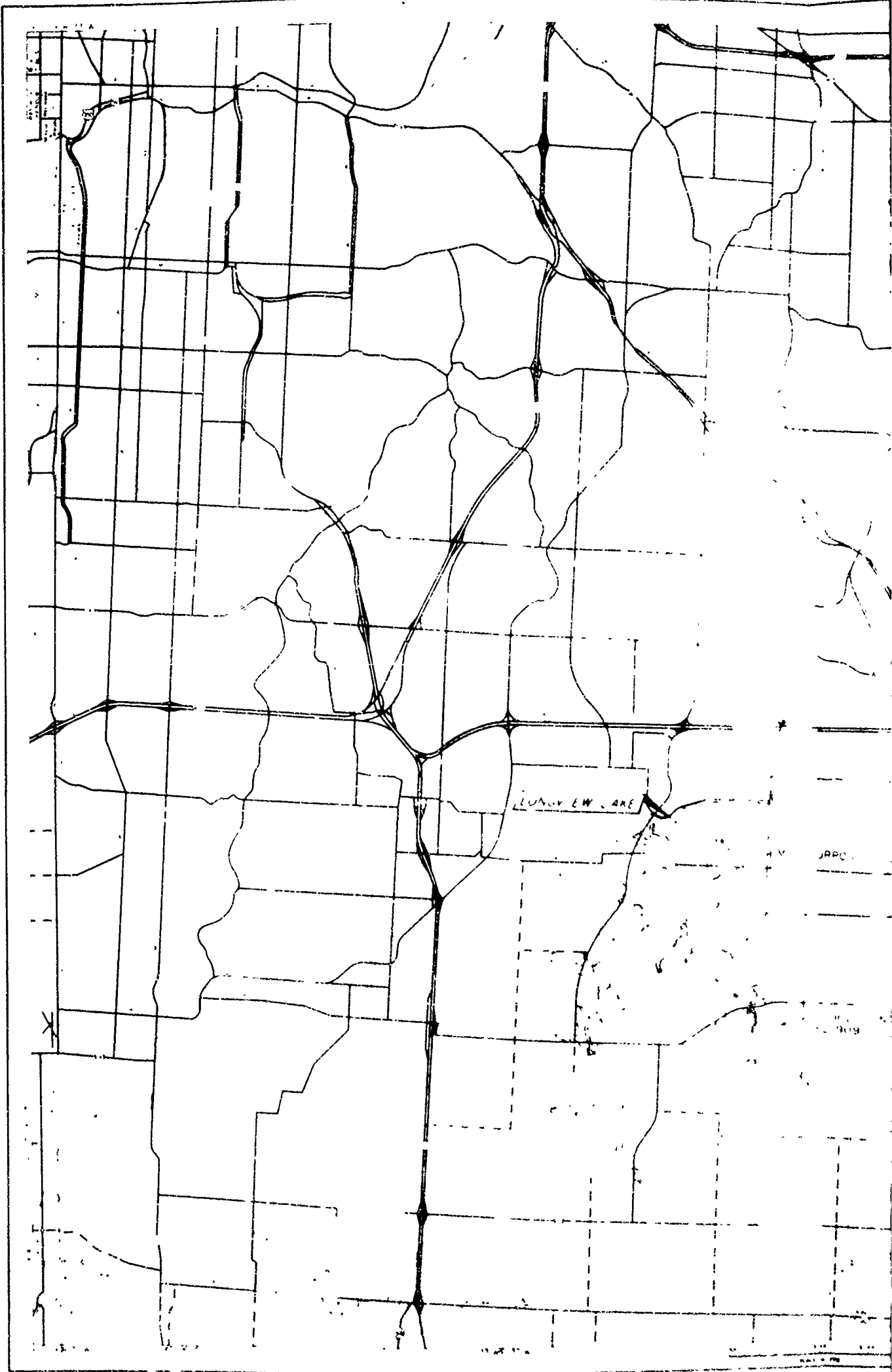


75. March 1990. Completed emergency rock stockpile on downstream left abutment. Looking northwest.

DRAWINGS

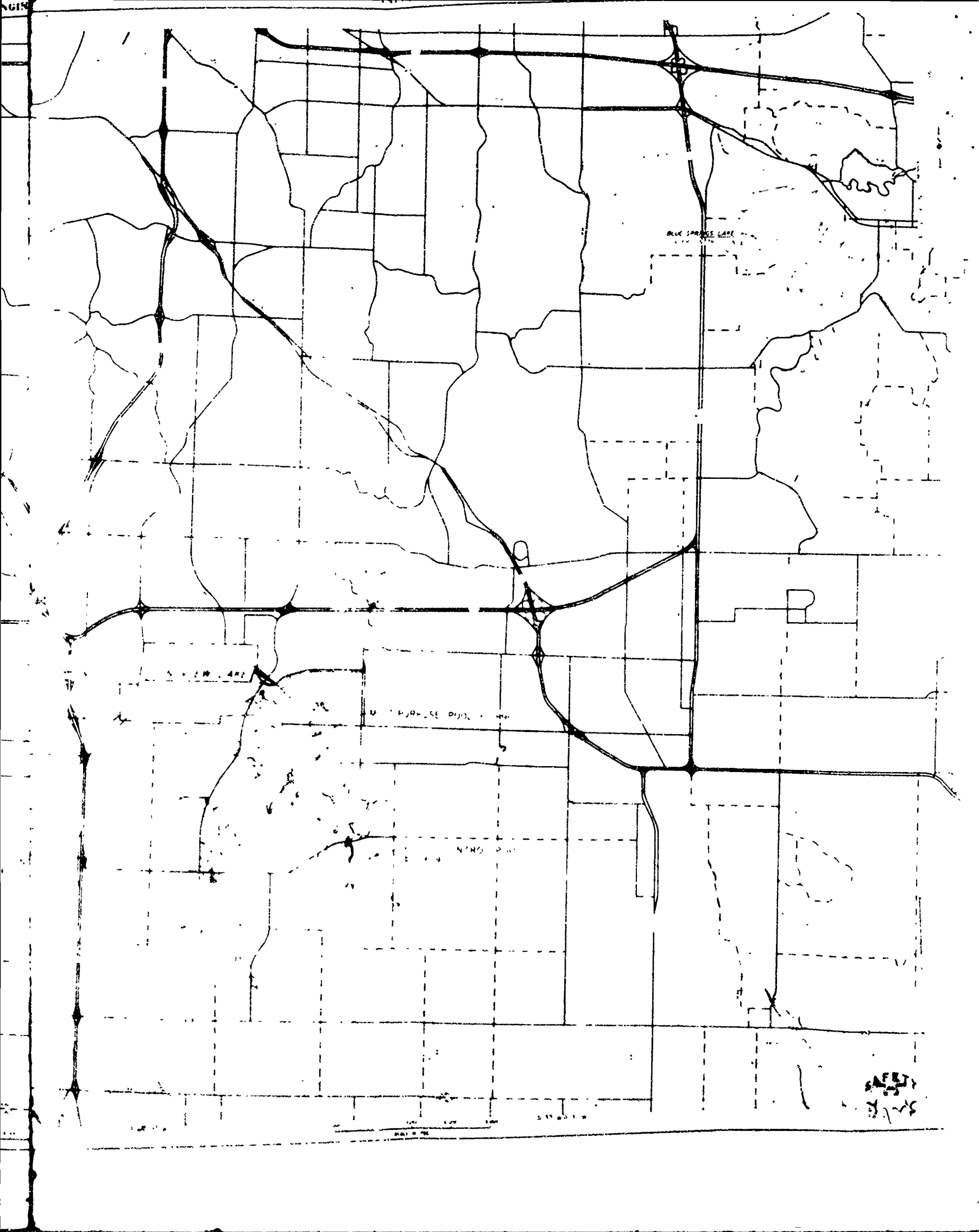
DRAWINGS

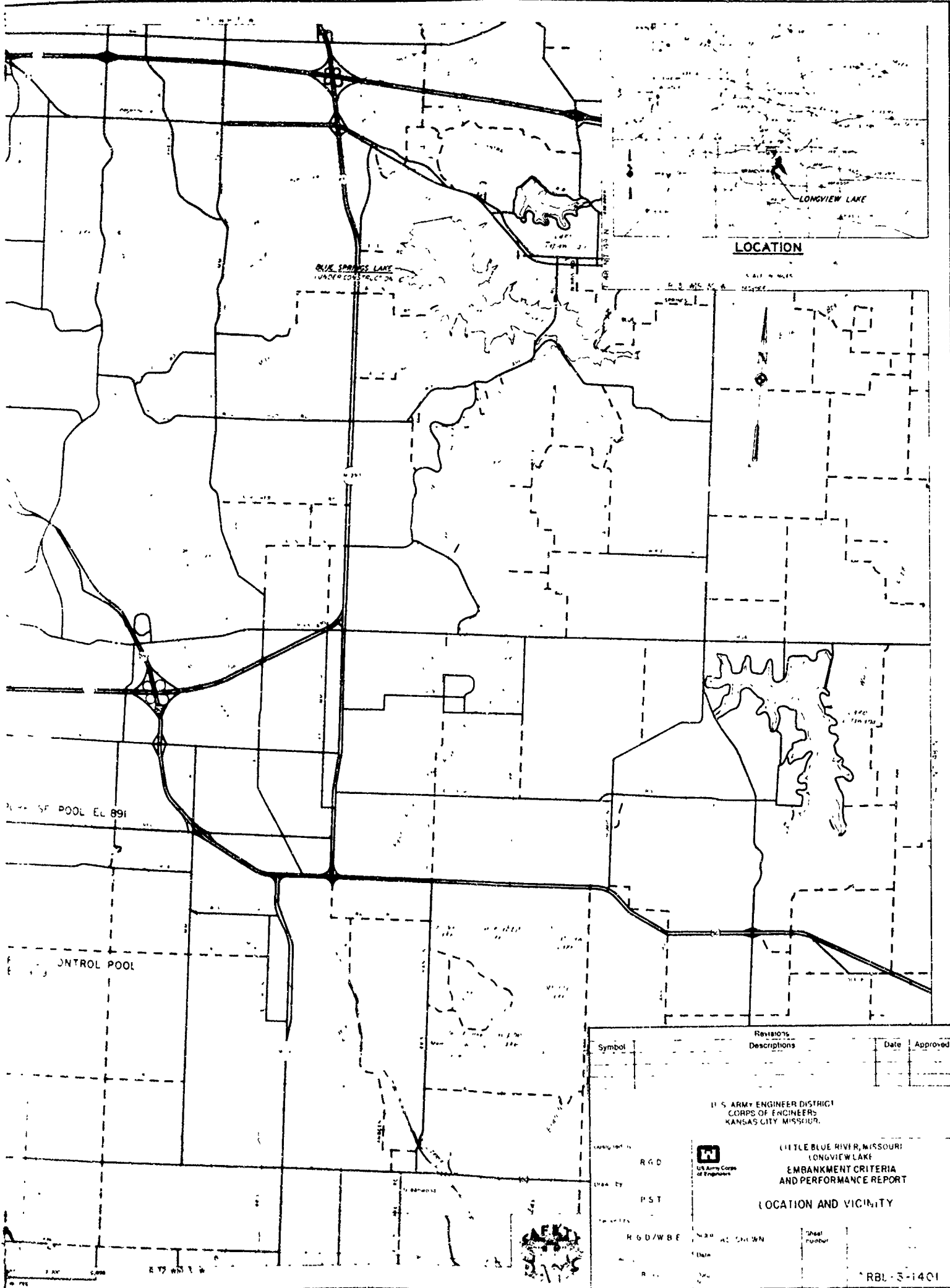




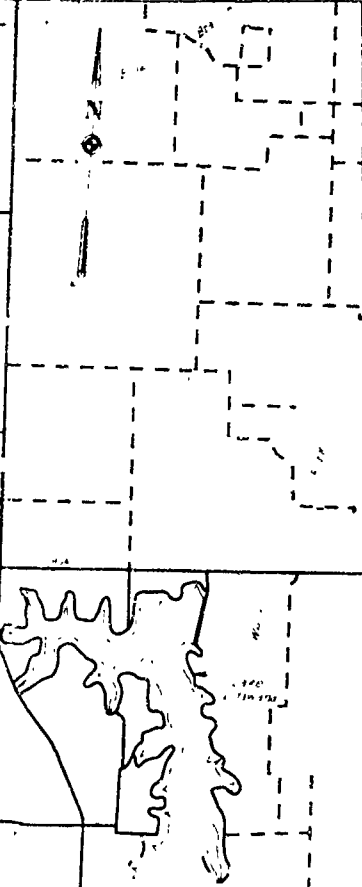
JRPO

1909





LOCATION



Symbol	Revisions Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

LOCATION AND VICINITY

REVISIONS  
1. RGD  
2. PST  
3. RGD/WBE

DATE  
11/1/50

BY  
H. L. CHILTON

Sheet  
Number

RBL-S-1401



ENGINE

BLUE

2.5

5 R2+12

1900 R14

925 R0+3

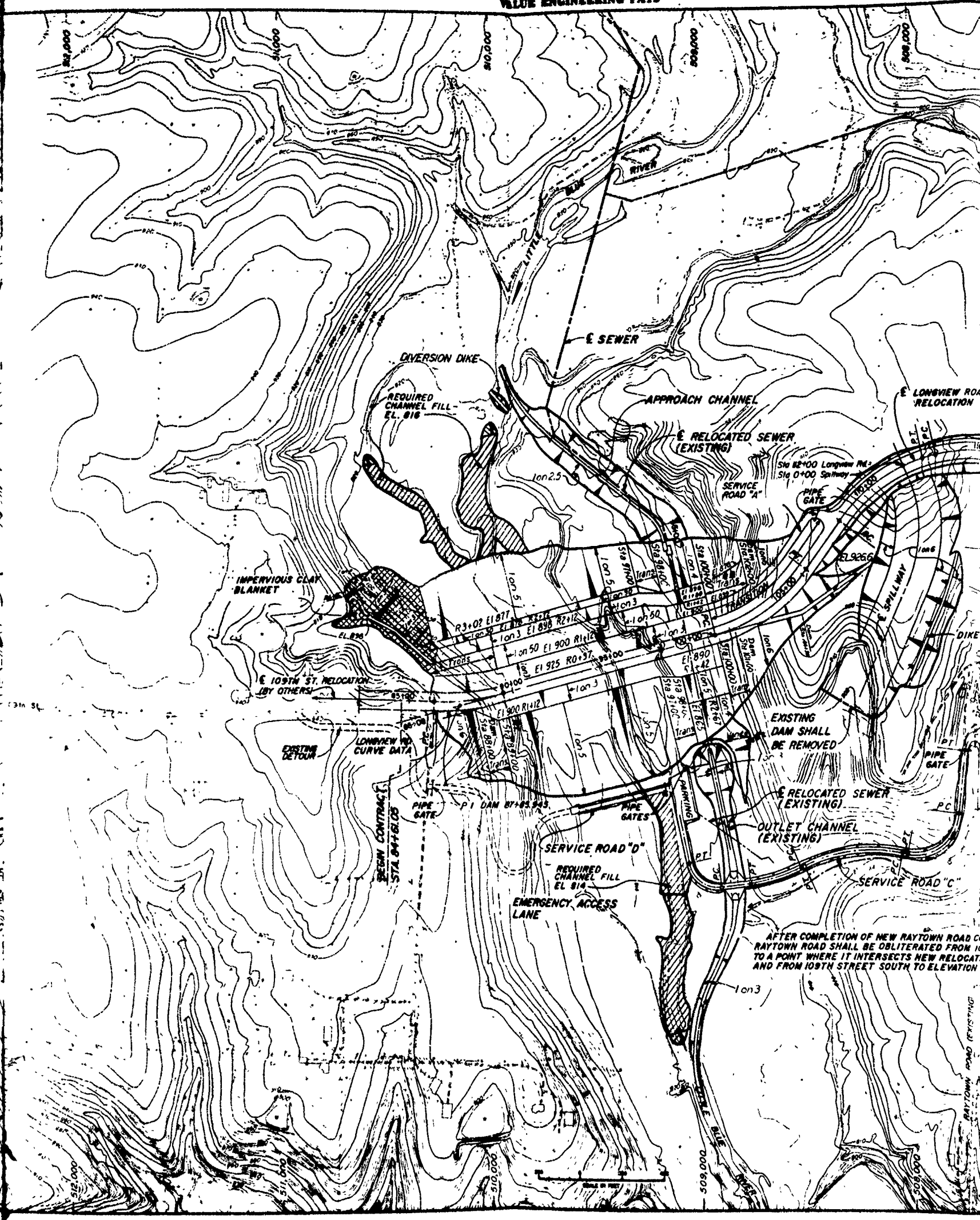
SERVICE

REQUIR

CHANNE

EL. 814

ENCY AC



DIVERSION DIKE

REQUIRED CHANNEL FILL  
EL. 816

E SEWER

APPROACH CHANNEL

E RELOCATED SEWER  
(EXISTING)

E LONGVIEW ROAD  
RELOCATION

SERVICE ROAD "A"

PIPE GATE

IMPERVIOUS CLAY  
BLANKET

E 109TH ST. RELOCATION  
(BY OTHERS)

EXISTING  
DETOUR

LONGVIEW RD  
CURVE DATA

PIPE GATE

P1 DAM BY 45.243

SERVICE ROAD "D"

REQUIRED CHANNEL FILL  
EL. 814

EMERGENCY ACCESS  
LANE

EXISTING  
DAM SHALL  
BE REMOVED

E RELOCATED SEWER  
(EXISTING)

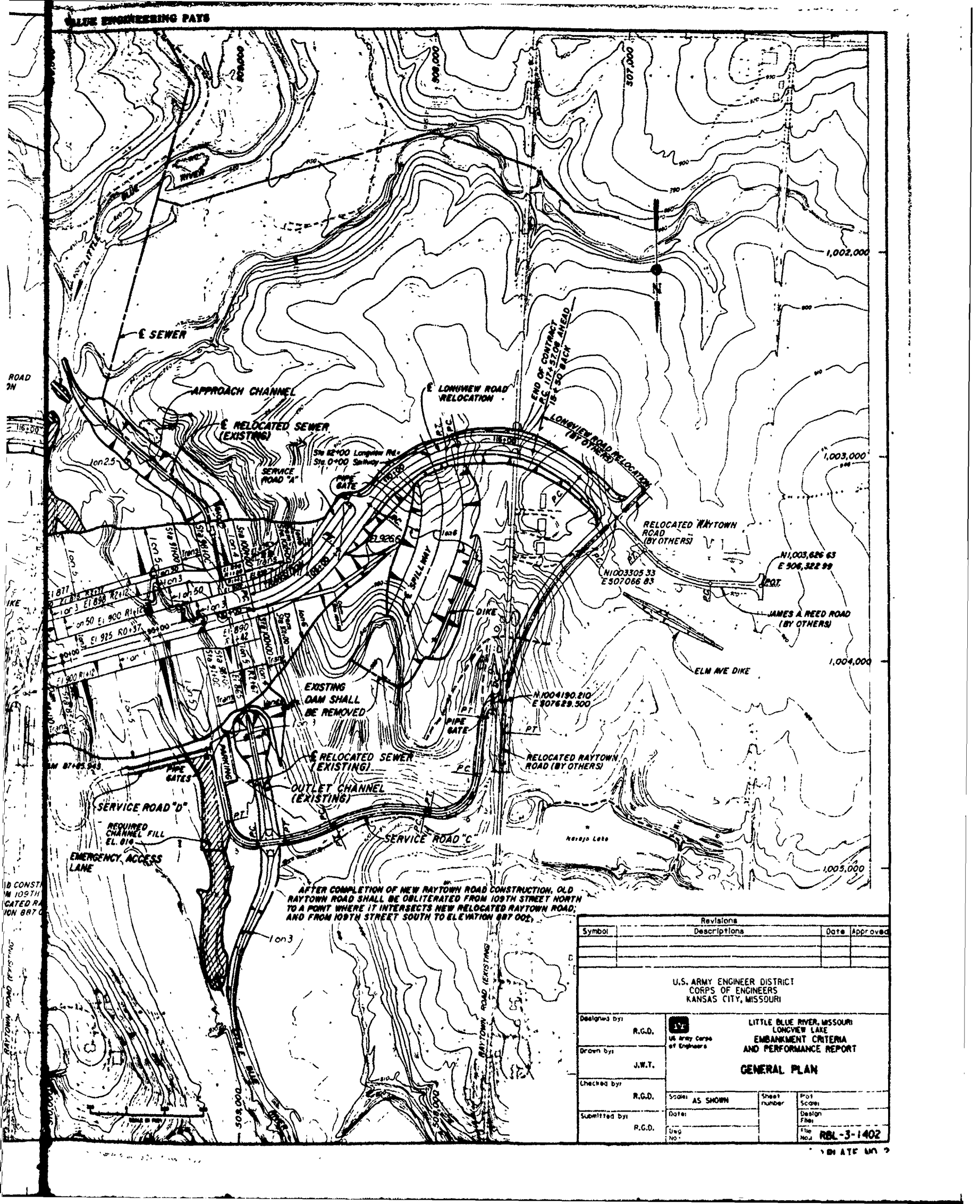
OUTLET CHANNEL  
(EXISTING)

SERVICE ROAD "C"

AFTER COMPLETION OF NEW RAYTOWN ROAD CON  
RAYTOWN ROAD SHALL BE OBLITERATED FROM 109  
TO A POINT WHERE IT INTERSECTS NEW RELOCATED  
AND FROM 109TH STREET SOUTH TO ELEVATION 81


SCALE 1" = 100'

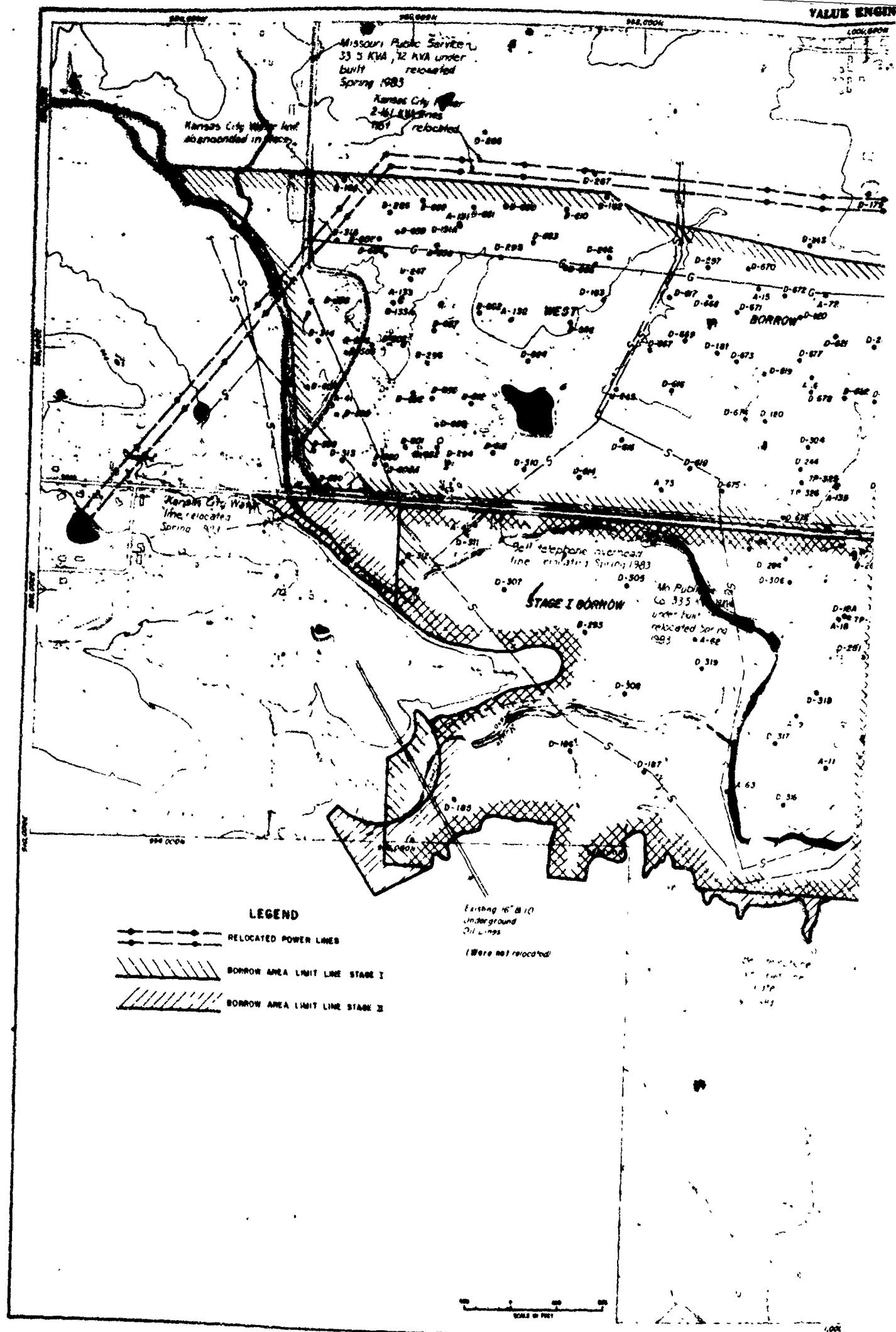
RAYTOWN ROAD (EXISTING)

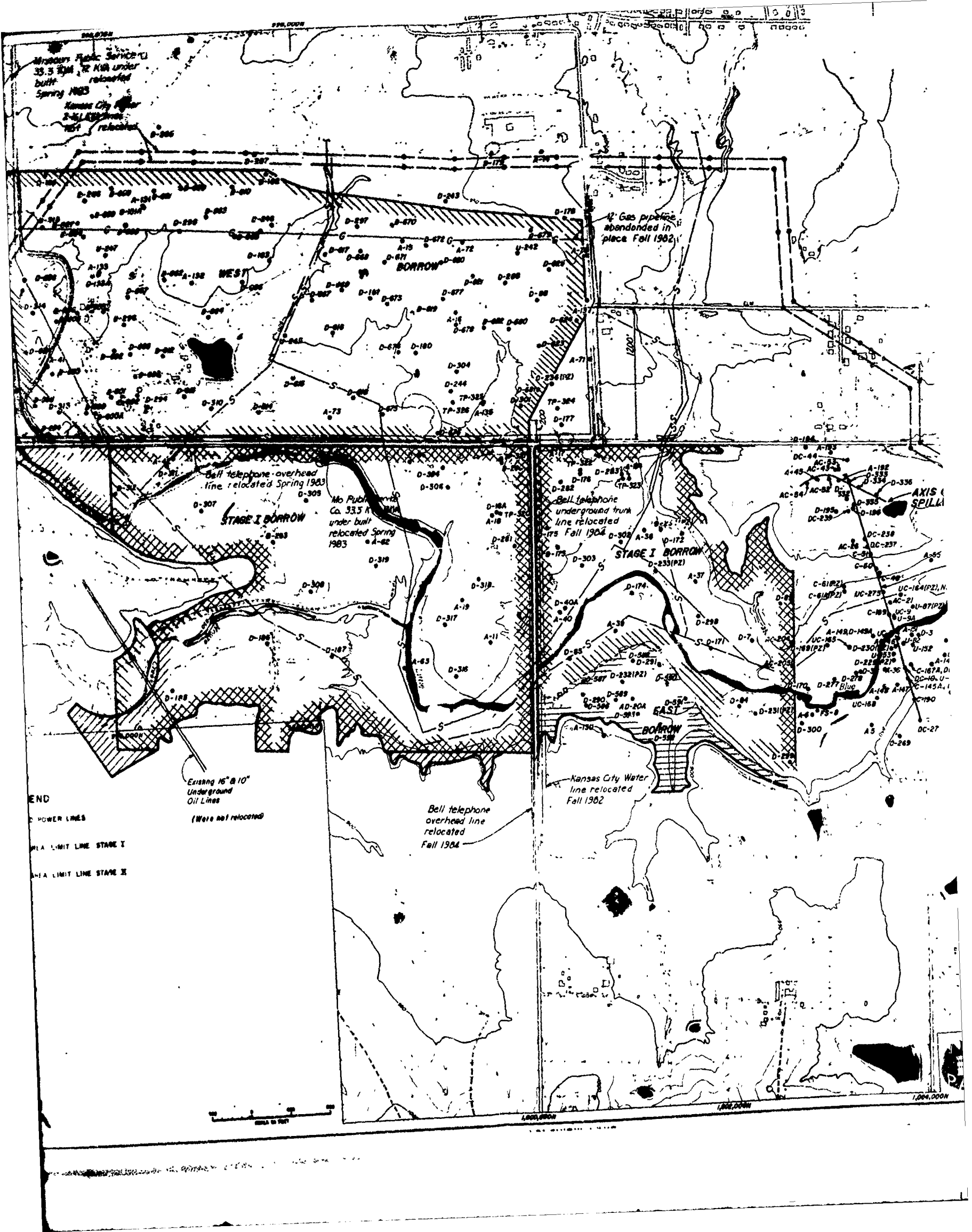


Revisions			
Symbol	Descriptions	Date	Approved

<p align="center">U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p>			
Designed by:  <div align="center">R.G.D.</div>	 U.S. Army Corps of Engineers	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by:  <div align="center">J.W.T.</div>	<div align="center"><b>GENERAL PLAN</b></div>		
Checked by:  <div align="center">R.G.D.</div>	Scale: AS SHOWN	Sheet number	Plot Scale:  Design File:  File No: <b>RRL-3-1402</b>
Submitted by:  <div align="center">R.G.D.</div>	Date:  Used No.		





Missouri Public Service Co.  
33.5 KVA 12 KVA under  
built  
Spring 1983  
Kansas City Power  
2-45 KVA under  
built  
relocated

12" Gas pipeline  
abandoned in  
place Fall 1982

Bell telephone overhead  
line relocated Spring 1983

STAGE I BORROW

Mo. Public Service  
Co. 33.5 KVA 12 KVA  
under built  
relocated Spring  
1983

Bell telephone  
underground trunk  
line relocated  
Fall 1984

STAGE I BORROW

AXIS  
SPILL

Existing 16" & 10"  
Underground  
Oil Lines  
(Were not relocated)

Bell telephone  
overhead line  
relocated  
Fall 1984

Kansas City Water  
line relocated  
Fall 1982

STAGE I BORROW

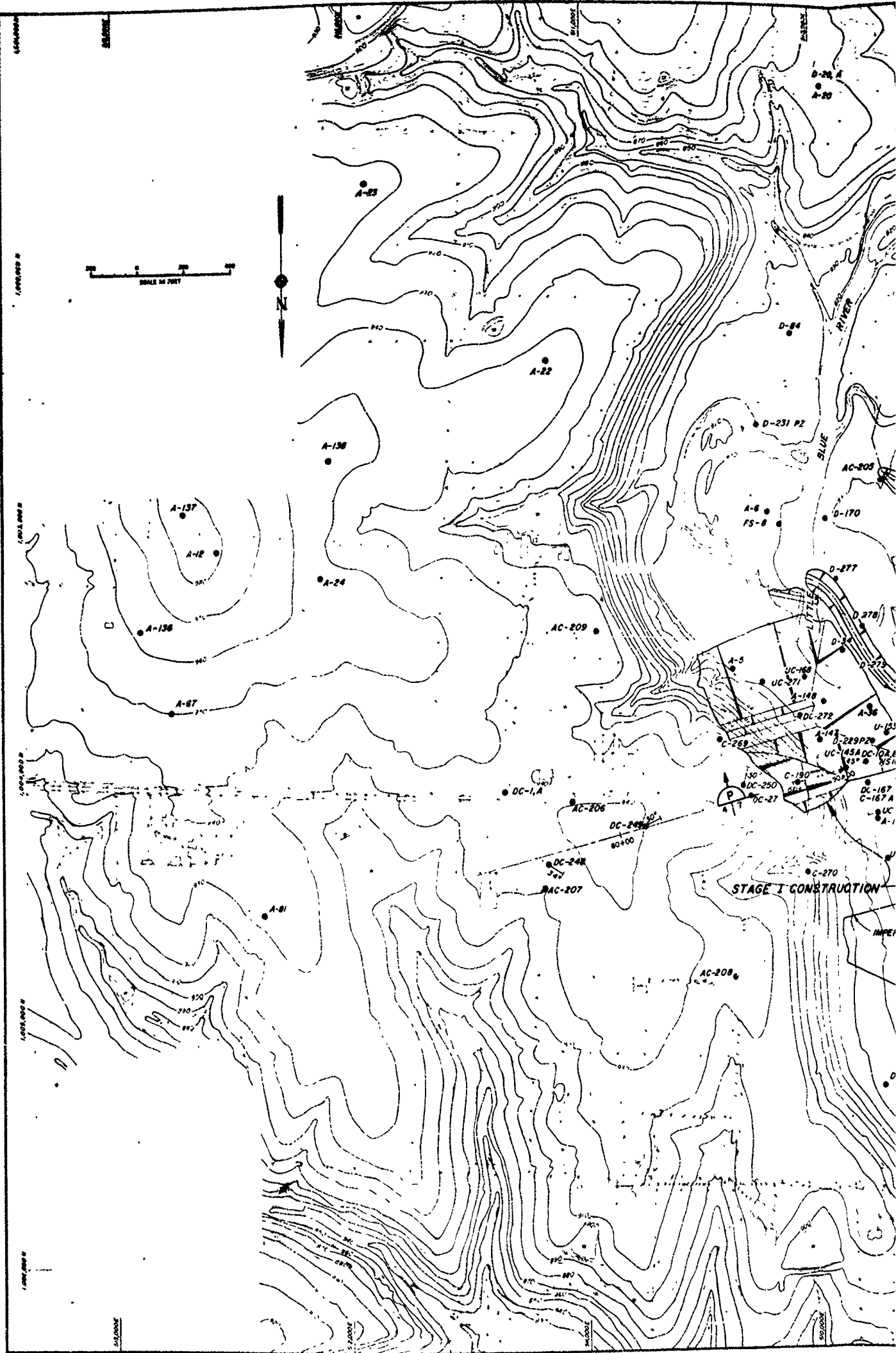
END  
POWER LINES  
LIMIT LINE STAGE I  
LIMIT LINE STAGE II

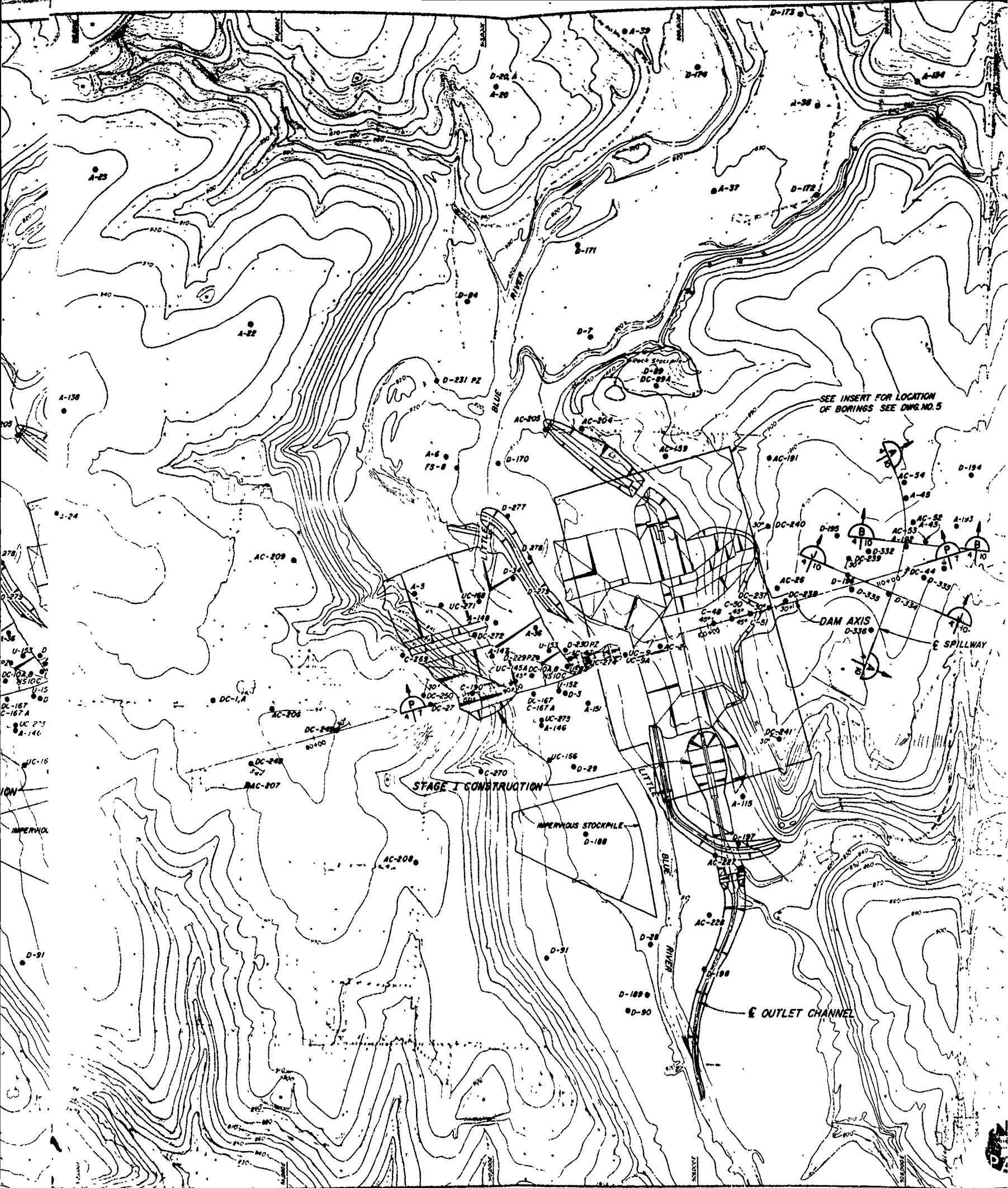
SCALE IN FEET

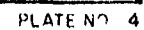
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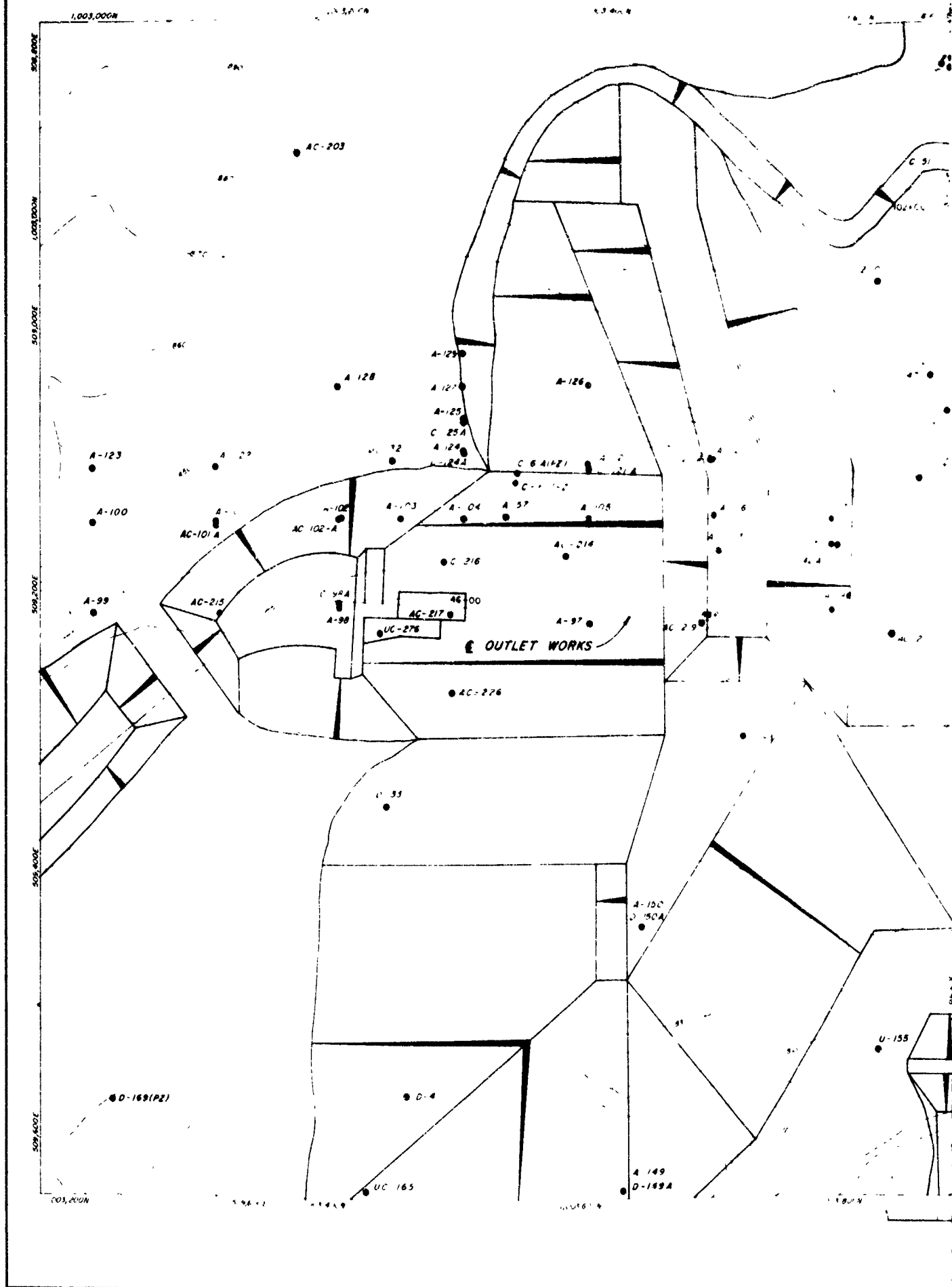


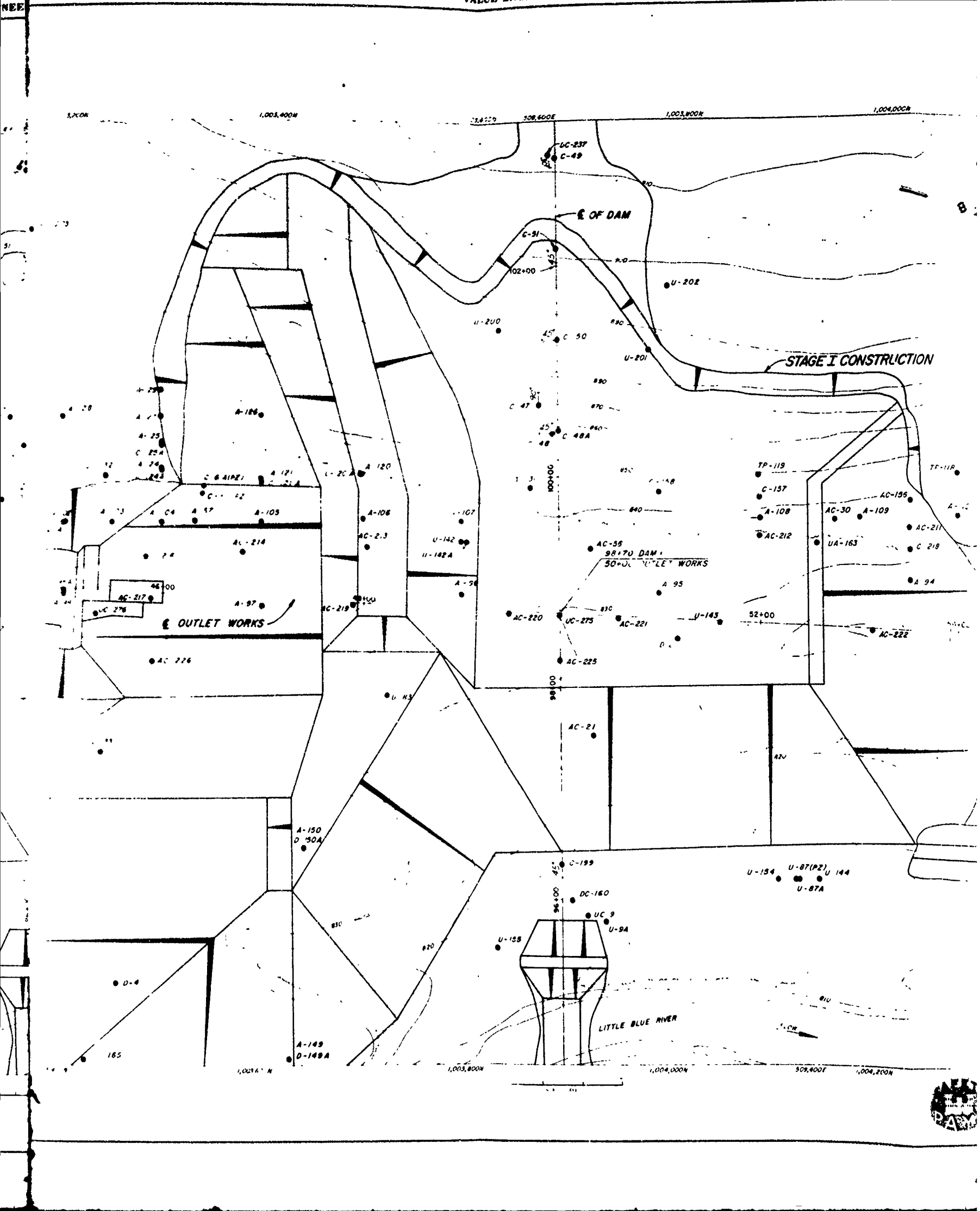














File No **RBL-3-1405**

GENERAL GEOLOGIC COLUMN					
SYSTEM	GROUP	FORMATION	MEMBER OR ZONE	SYMBOL	GENERALIZED DESCRIPTION
TRANS-JANA	KAMAS F.F.	HERBERTVILLE	WESTERVILLE	W	Limestone, massive, gray, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOA	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
		DUBA	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
		CALLEMAN	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
		SOUTH	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
	HERBERTVILLE	DOCK	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
		DOCK	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
		DOCK	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
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			DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
	DOCK	DOCK	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.
		DOCK	DOCK	D	Small, dark gray, shaly, with thin, dark gray, shaly parting. Thin, dark gray, shaly parting.



## HC COLUMN

#### GENERALIZED DESCRIPTION

1. medium to coarse bedded medium to dark bedded micaceous gray gray

2. thin clayey clayey to silty calcareous dark gray

3. thin 2-3' of medium micaceous hard dense to heavy crystalline due to

4. thin gray with occasional dark gray shale lenses and beds

5. very crystalline medium to thick bedded micaceous argillaceous

6. the body of dark gray shale

7. in the middle dark gray with micaceous nodules common in local

8. crystalline thin to medium bedded argillaceous poorly micaceous

9. shale springs and occasional thin beds of dark gray shale

10. characteristic micaceous beds near part a gray clay to black

11. thin layer calcareous near bottom

12. hard to soft crystalline thin to thick bedded micaceous grayish

13. with the partings of gray shale 1 to 2' massive micaceous in

14. gray micaceous dark gray lower part is soft less calcareous

15. gray and gray shales

16. thin to heavy crystalline thin to medium bedded argillaceous medium

17. gray to black shale

18. thin to medium part medium gray

19. gray to very heavy crystalline thick bedded argillaceous gray gray

20. with some partings

21. thin to medium gray to dark gray

22. gray to medium gray gray with micaceous nodules

23. lower part is micaceous fine argillaceous sandy thin to

24. medium thick bedded to massive argillaceous micaceous gray

25. gray to black to very micaceous with medium gray to black

26. gray to micaceous

27. medium dark gray to gray and to some thin bedded to massive

28. thin to heavy argillaceous to medium to massive micaceous

29. thin to medium gray to black to massive micaceous

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## UNIFIED SOIL CLASSIFICATION SYSTEM

GW	Well-graded gravels, gravel-sand mixtures, little or no fines	ML	Inorganic silts and silts, fine sands from river, silts or clayey, low sand or clayey silts in slight plasticity
GP	Poorly graded gravels or gravel-sand mixtures, little or no fines	CL	Inorganic clays of low to medium plasticity, gravelly c-s, sandy clays, silty clays, lean clays
GM	Silty gravels, gravel-sand mixtures	OL	Organic silts and organic silty clays of low plasticity
GC	Clayey gravels, gravel-sand mixtures	MLH	Inorganic silts, micaceous or siltaceous, fine sandy or silty soils, elastic silts
SW	Well-graded sands, gravelly sands, little or no fines	CH	Inorganic clays of high plasticity, fat clays
SP	Poorly graded sands or gravelly sands, little or no fines	OH	Organic clays of medium to high plasticity, organic silts
SM	Silty sands, sand-silt mixtures	PT	Peat and other highly organic soils
SC	Clayey sands, sand-clay mixtures		

Classification from actual laboratory tests where LL and PI are shown.  
Dust classification where used is in accordance with the United Soil Classification System.  
For details of the United Soil Classification System See Waterways Experiment Station  
Technical Memorandum No. 3357 dated March 1963 and revised in 1960

TERM FOR CONSISTENCY OF  
SOIL AND HARDNESS OF BEDROCK

CVL

Concrete (10000) Ultimate Uniaxial Compressive Strength  
Tens per square inch

10000	~ 3200
5000	025 05
4000	05 10
3000	10 20
2000	20 40
1000	> 40

BEDROCK  
SCALE OF HARDNESS

Very soft plastic	Can be scratched easily with thumb
Soft	Can be scratched with fingernail
Moderately hard	Can be scratched easily with knife
	Cannot be scratched with fingernail
Very hard	Can scratch with knife
Very very hard	Can be scratched with a file

LEGEND FOR L-5

Physic's name \_\_\_\_\_  
 Location of observation \_\_\_\_\_  
 Date of report \_\_\_\_\_  
 \* Name of Station and Range & M. C. Connection \_\_\_\_\_  
 Name of Agent and Direction \_\_\_\_\_  
 Division of (Outfitted) Marine \_\_\_\_\_

Moisture content - per core

$E_{\text{core}} = \frac{\text{wt} - \text{wt}_{\text{dry}}}{\text{wt}_{\text{dry}}} \times 100$

Water Depth encountered during drilling

United States classification determined as follows:

1. Sample

2. Core

Percent Lost Drill Water

Lost Core (Bedrock)

Drilling completion status

\* On a 100 ft. plot of section may be used to  
of Reedward (Land or River) or Right or Left  
The line is 100 ft. in length.

## MAP SYMBOL

- Vertical bracing
- Inclined bracing, in any direction and with 4:1 slope

## ABBREVIATIONS

[illegible]Dr  
W  
L

Symbol	Meaning
1	1
2	2
3	3
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100	100

Designed by

Drawn by

Checked by

**1**

Submitted by







MITTS

ROCK

EL. 65° E

ITS

mod ha

lylike

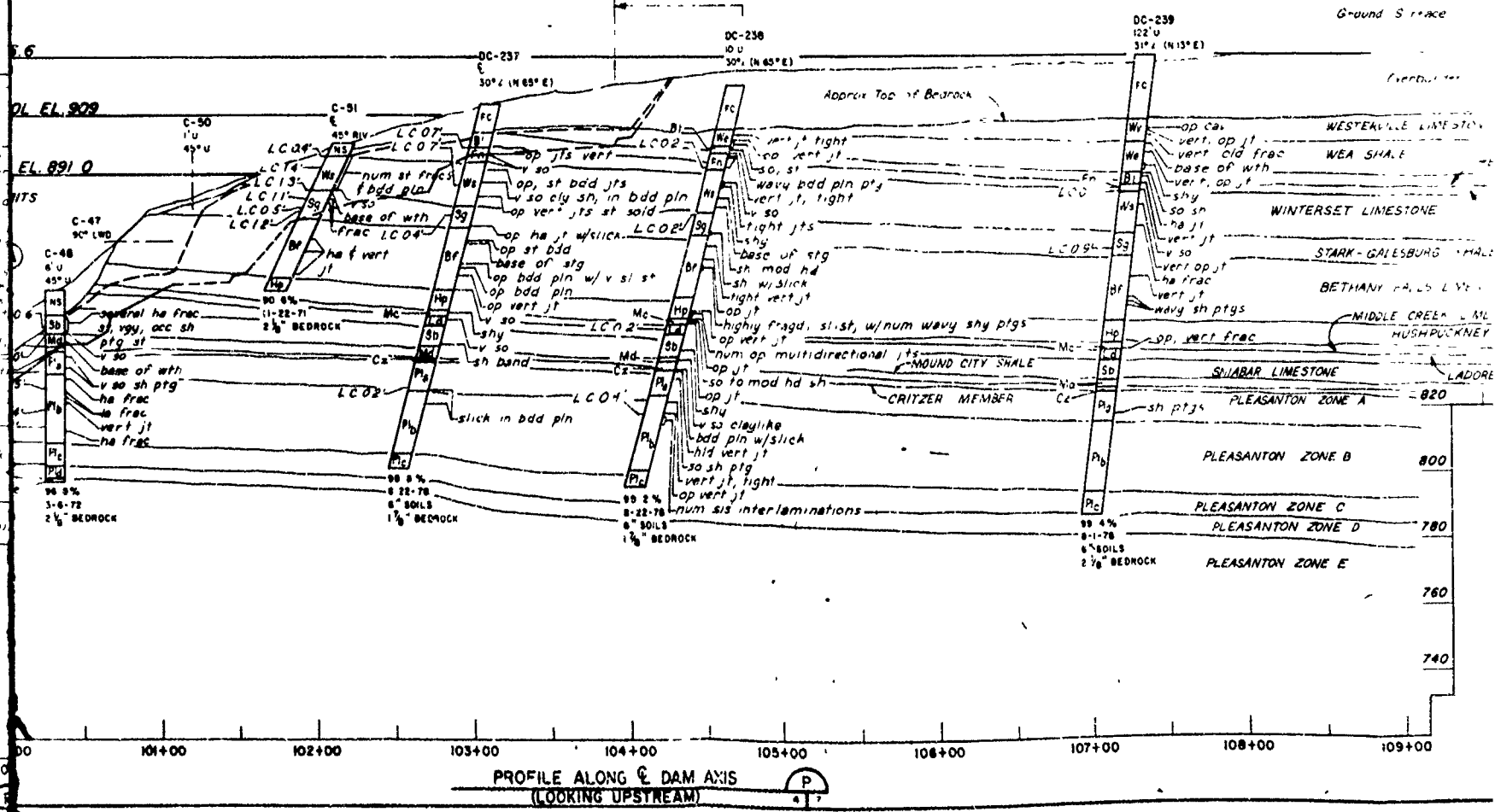
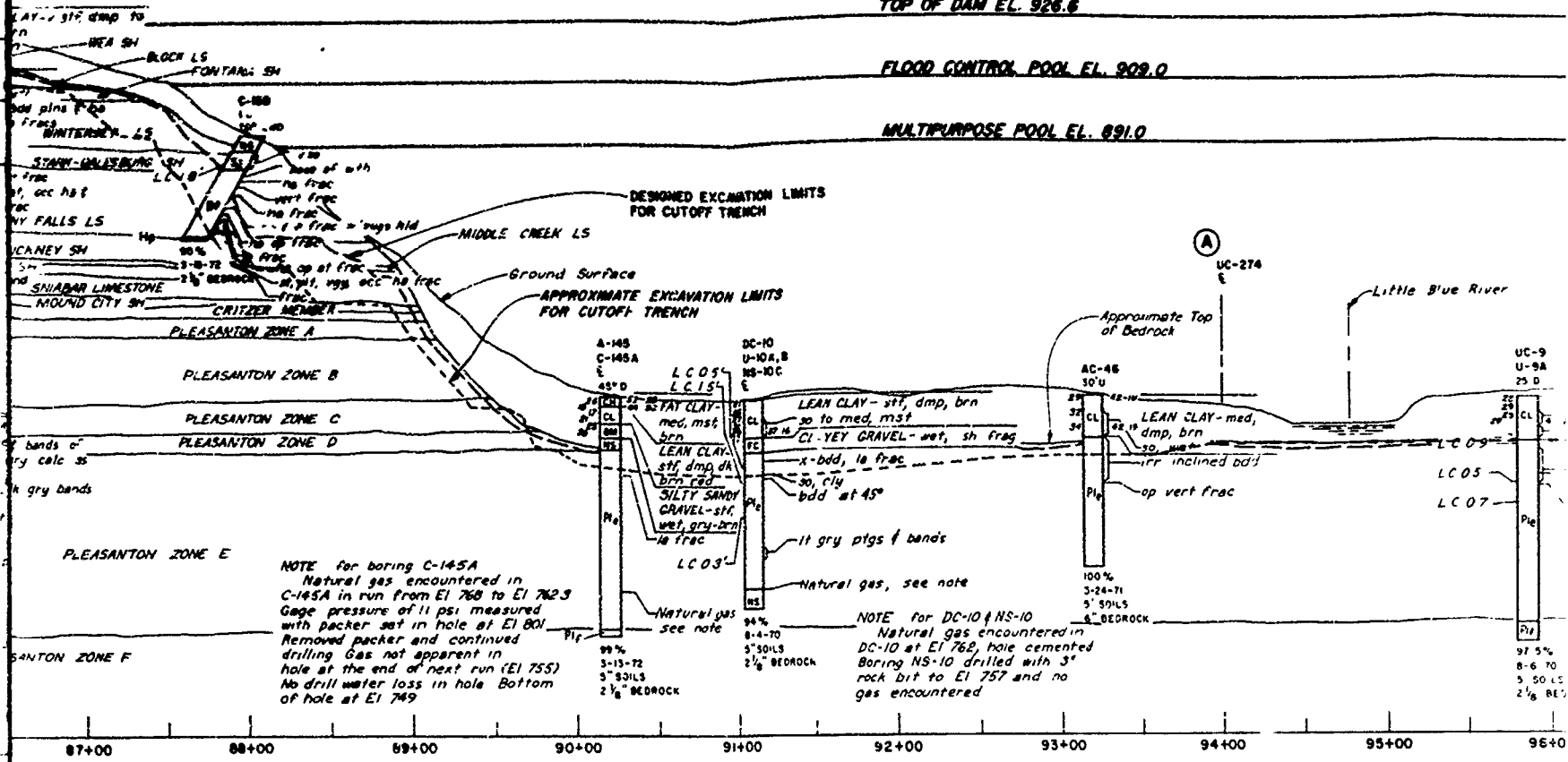
right

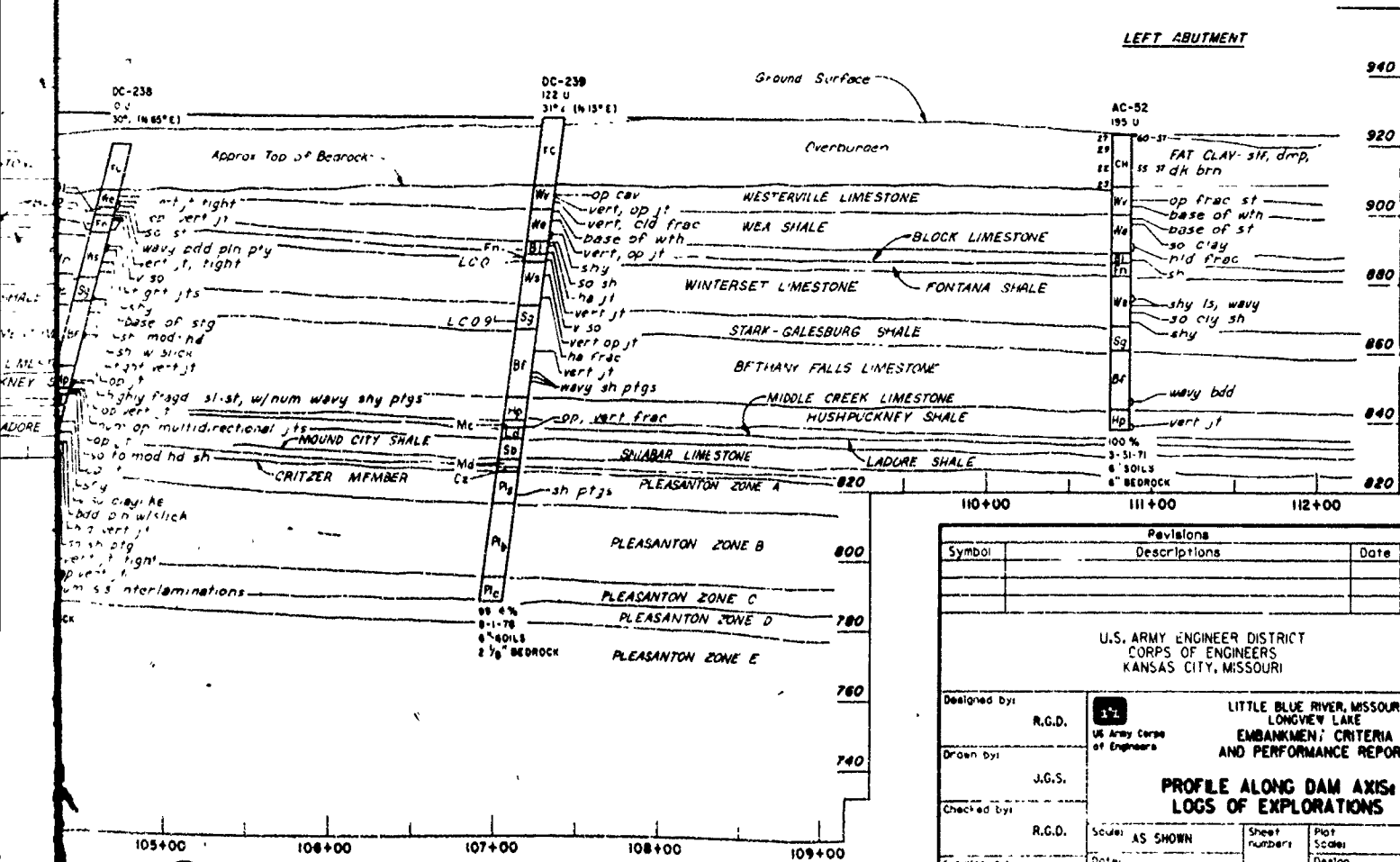
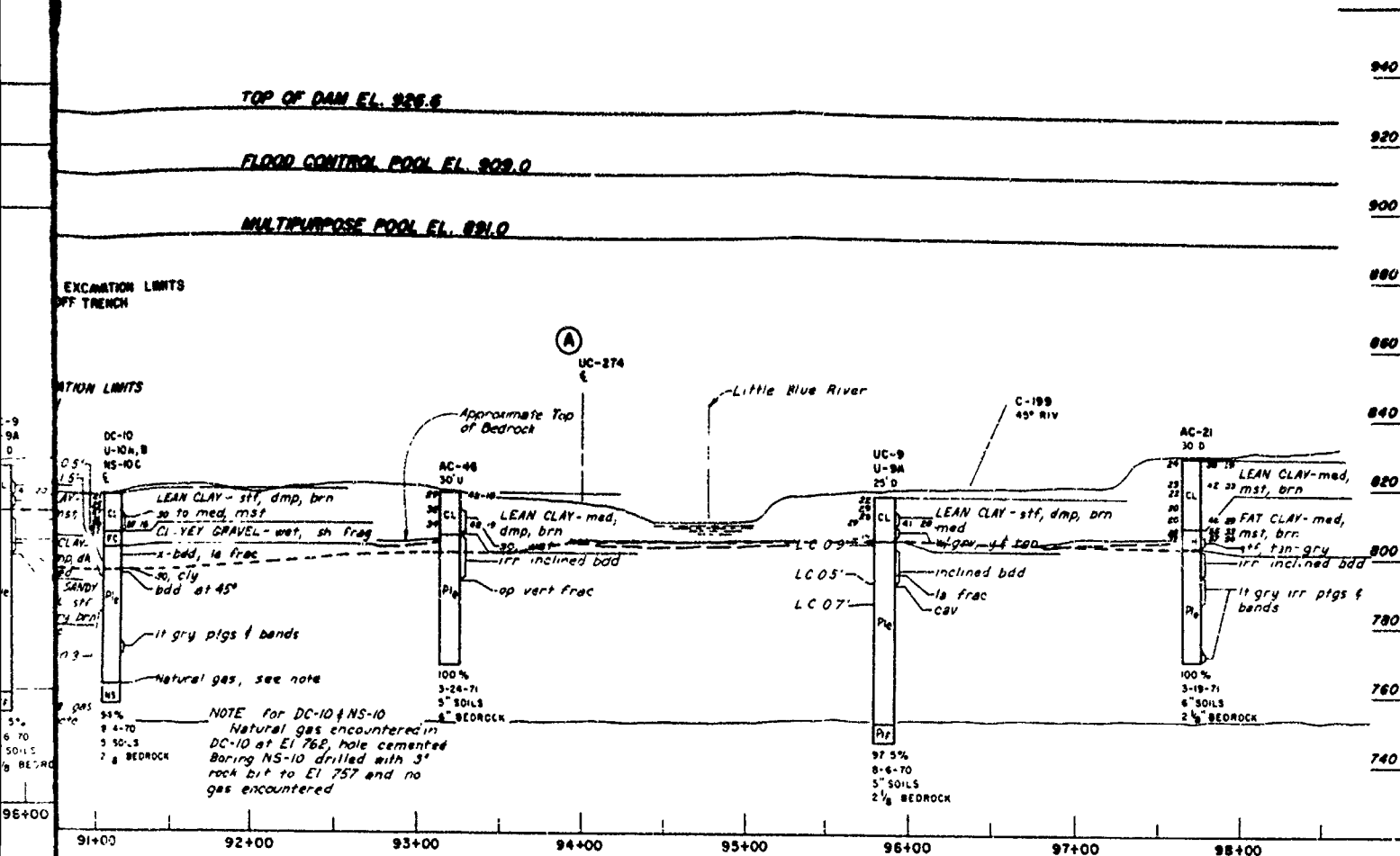
ter(lam)

TOP OF DAM EL. 926.6

FLOOD CONTROL POOL EL. 909.0

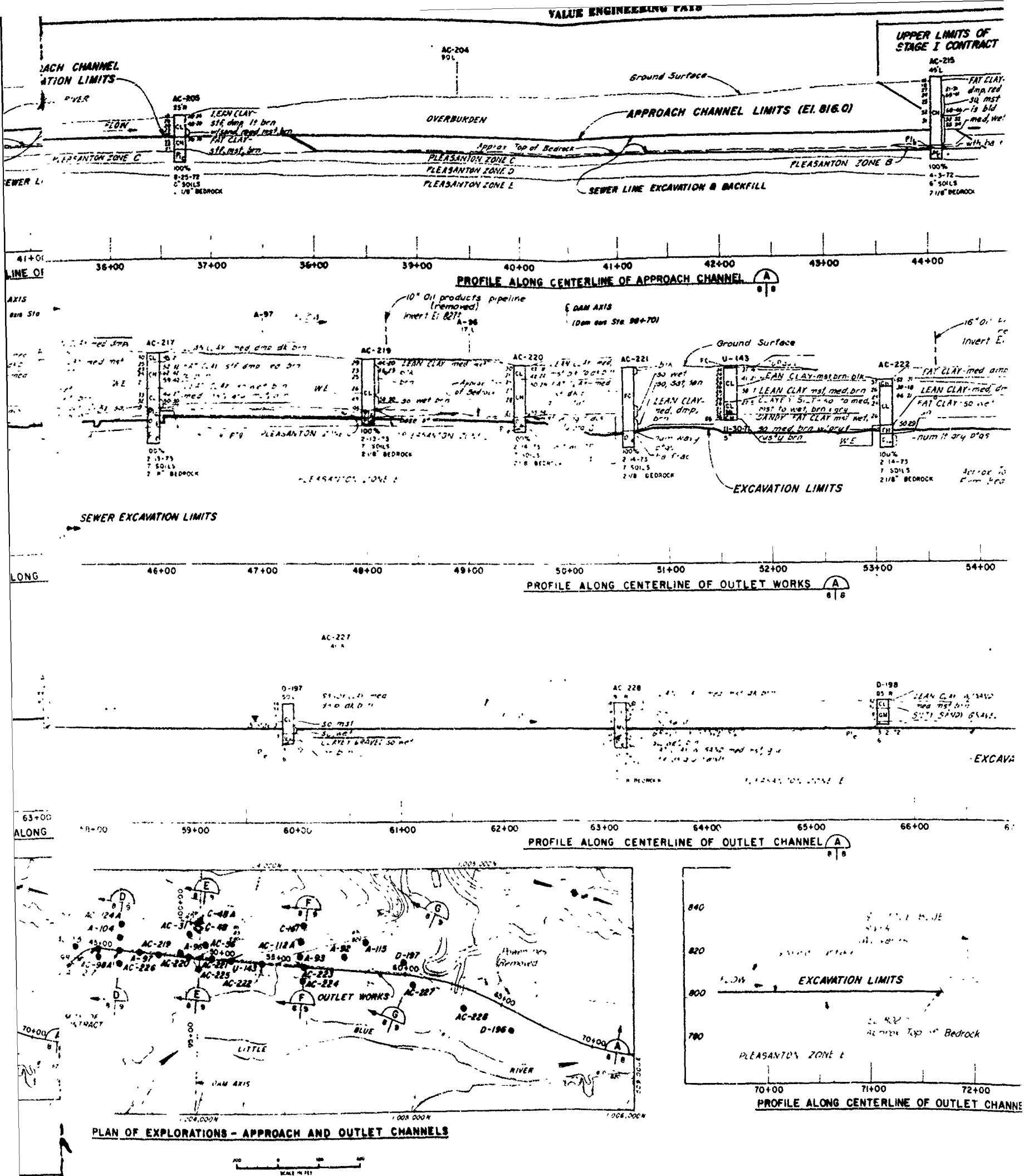
MULTIPURPOSE POOL EL. 891.0

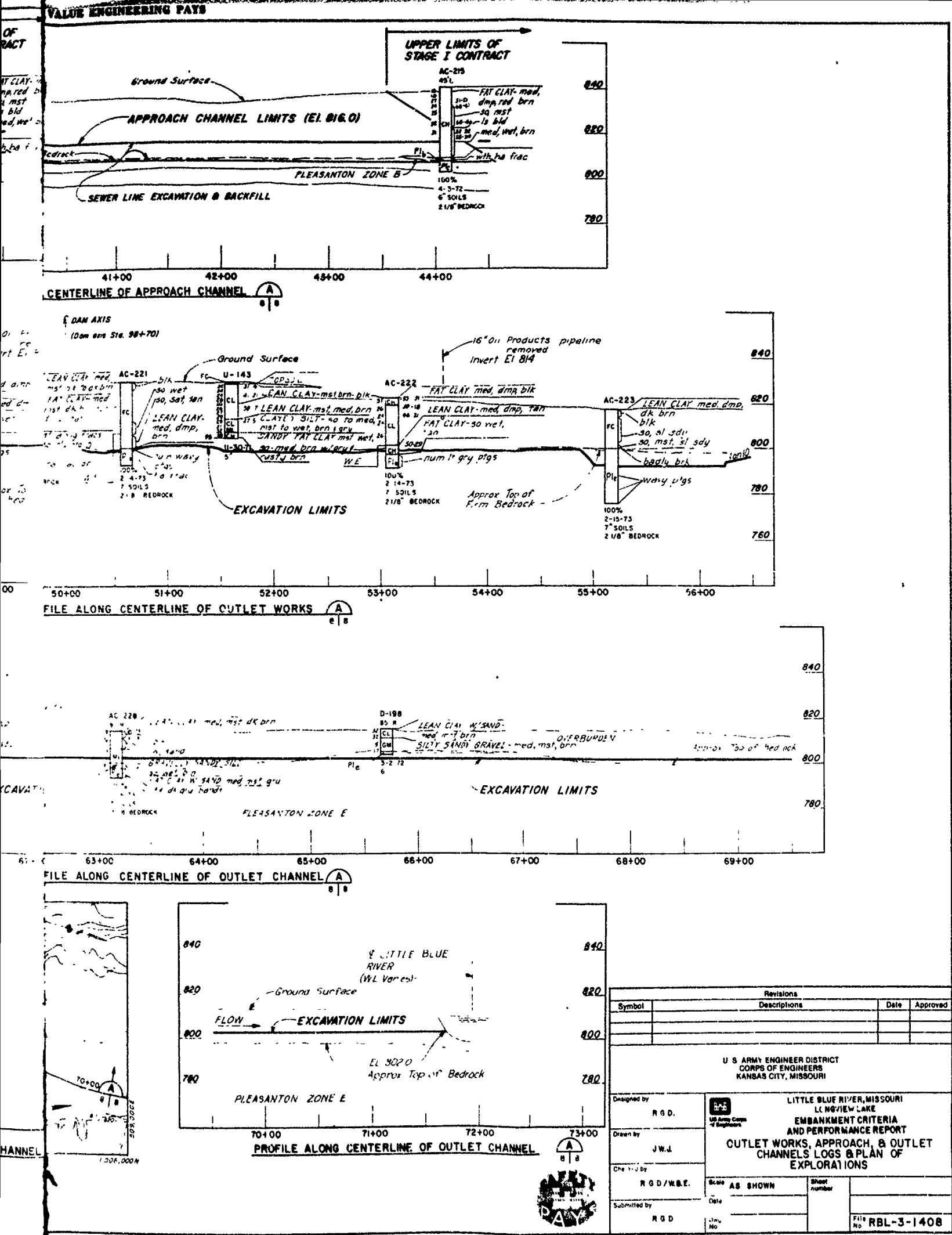




Revisions			
Symbol	Descriptions	Date	Approved
<p>U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p>			
Designed by:	R.G.D.	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</p>	
Drawn by:	J.G.S.	<p><b>PROFILE ALONG DAM AXIS LOGS OF EXPLORATIONS</b></p>	
Checked by:	R.G.D.	Scale: AS SHOWN	Sheet numbers:
Submitted by:	R.G.D.	Date:	Plot Scale:
		Drawn by:	Design File:
			File No:

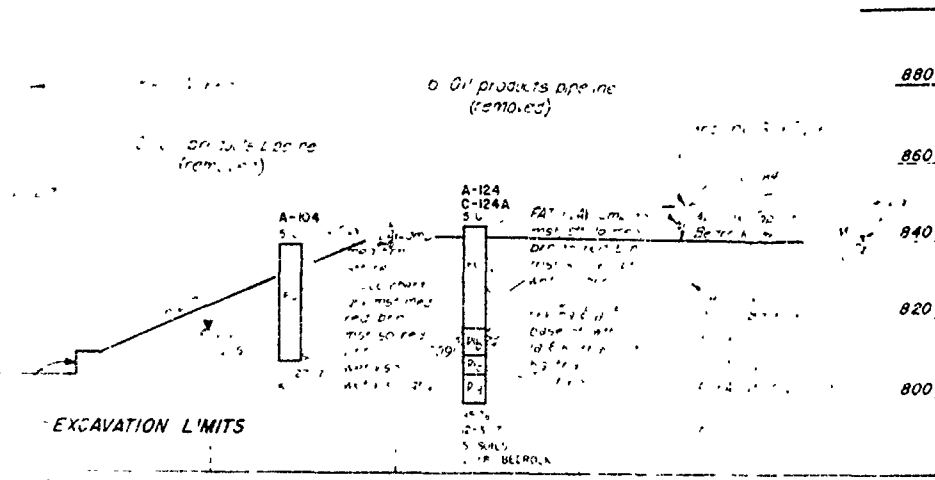




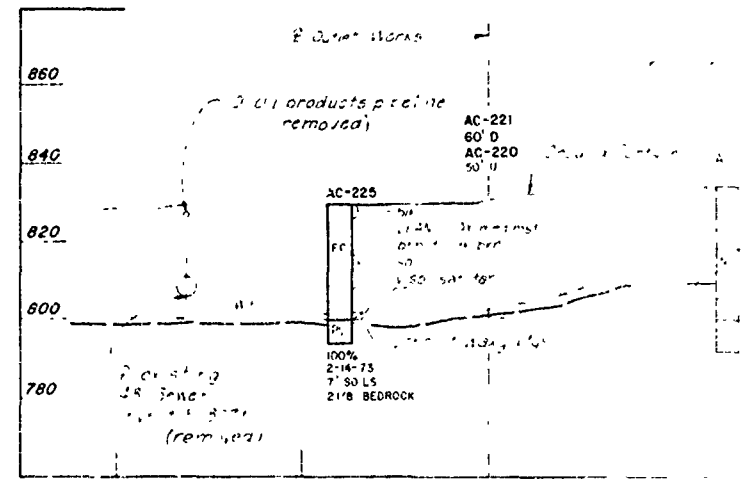




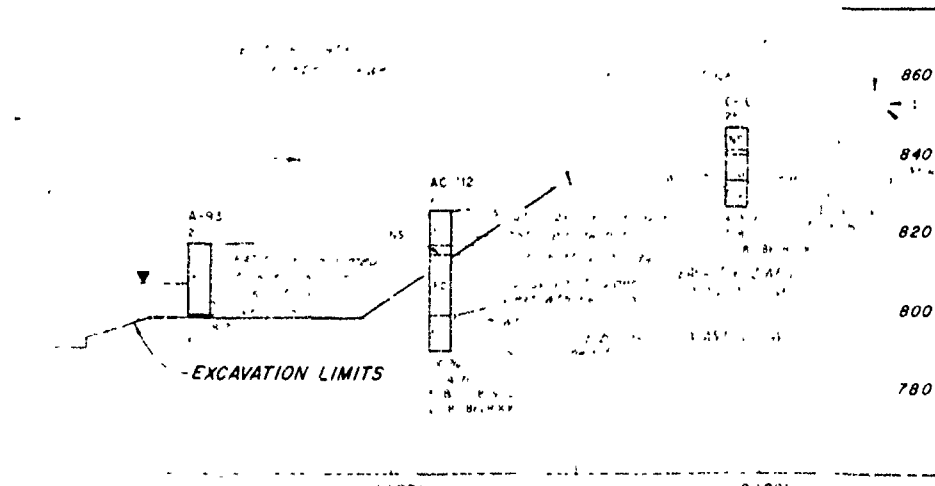




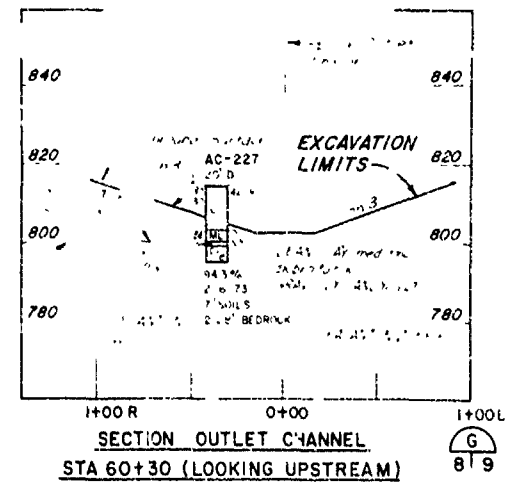
SECTION AT INTAKE TOWER-STA 46+00 (LOOKING UPSTREAM) D  
8/9



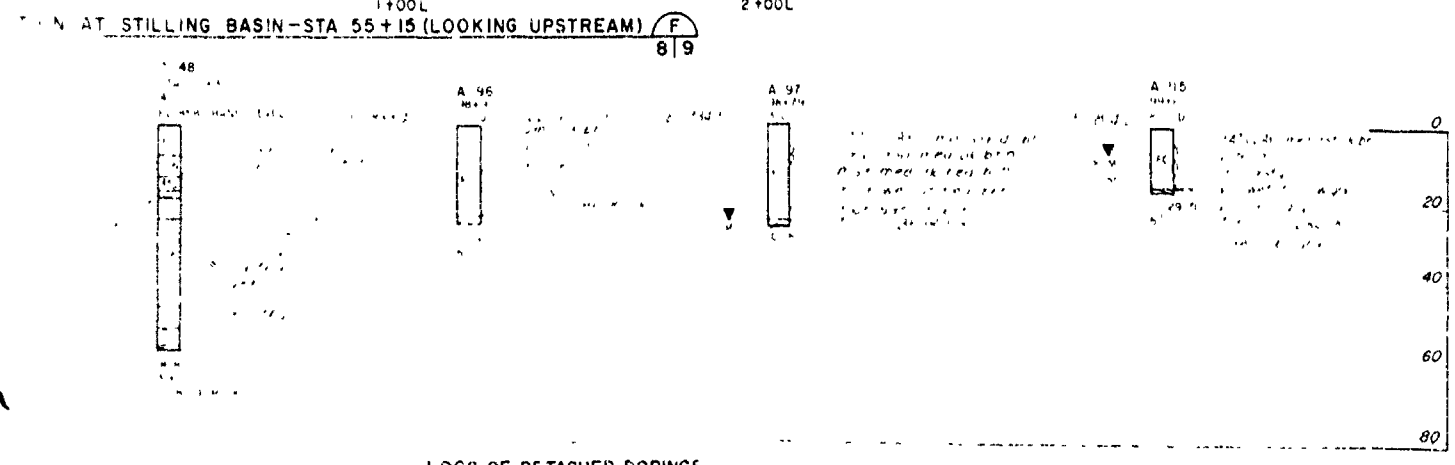
SECTION AT OUTLET WORKS-STA 0+00



SECTION AT STILLING BASIN-STA 55+15 (LOOKING UPSTREAM) F  
8/9

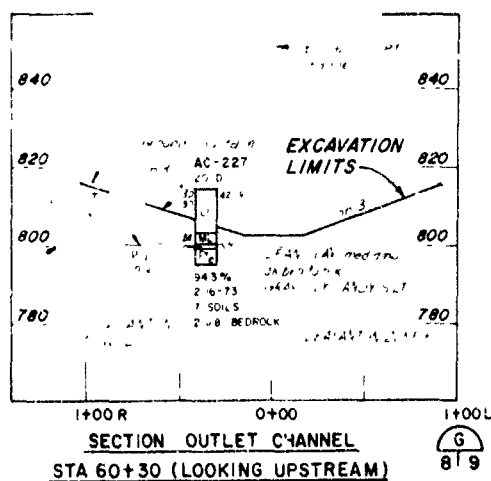
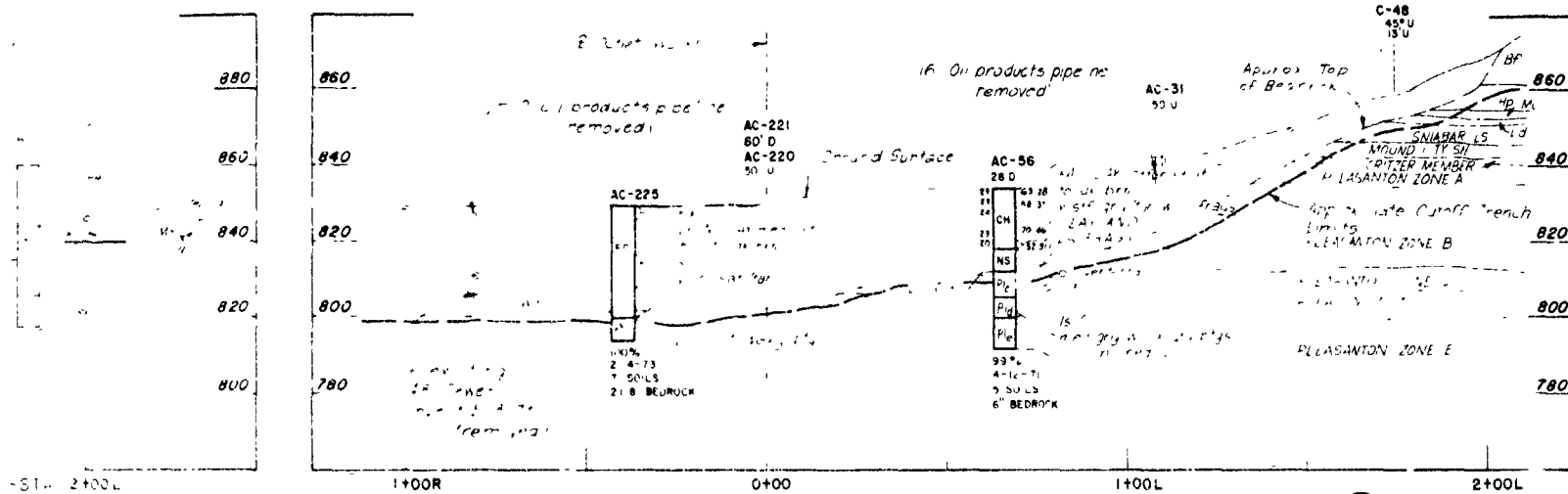


SECTION OUTLET CHANNEL  
STA 60+30 (LOOKING UPSTREAM) G  
8/9



LOGS OF DETACHED BORINGS

SAFETY  
8/9



Revisions			
Symbol	Descriptions	Date	Approved

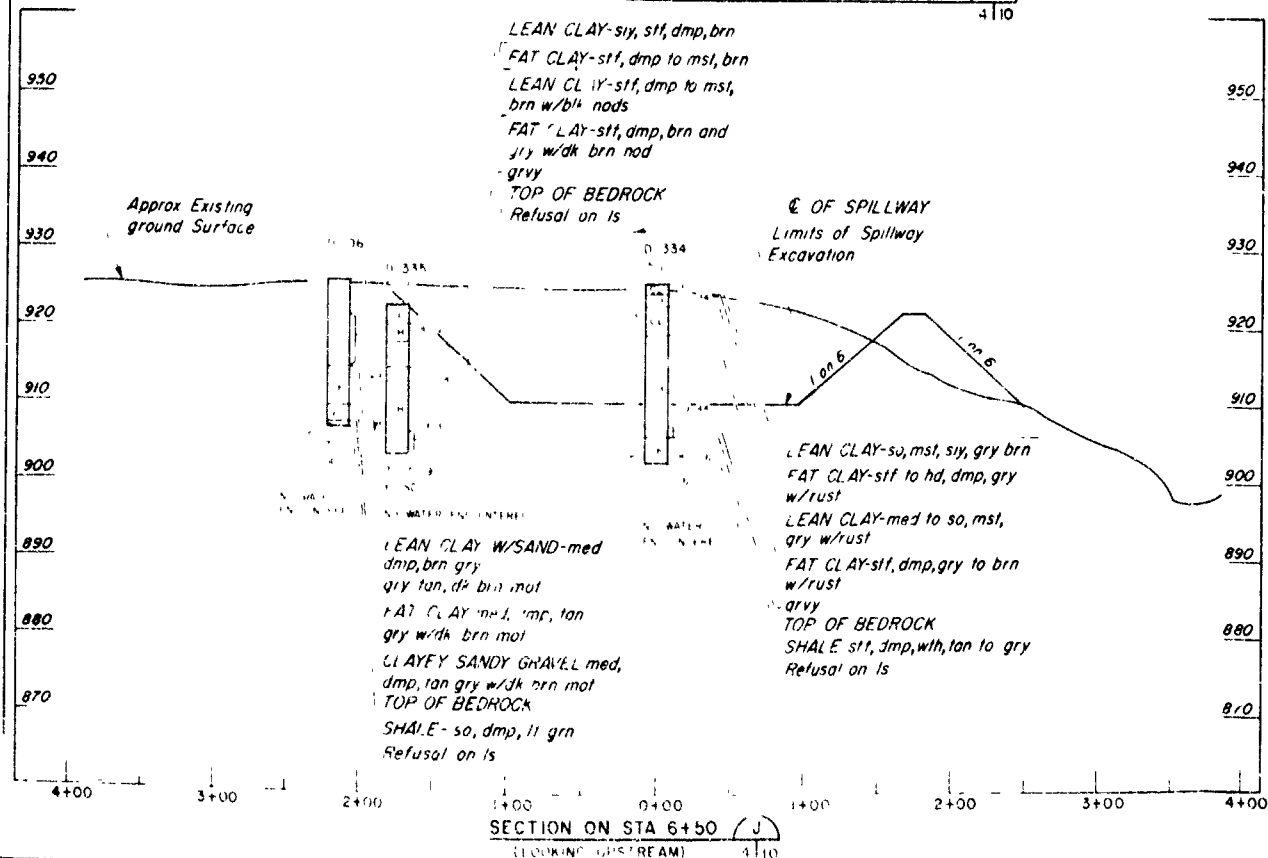
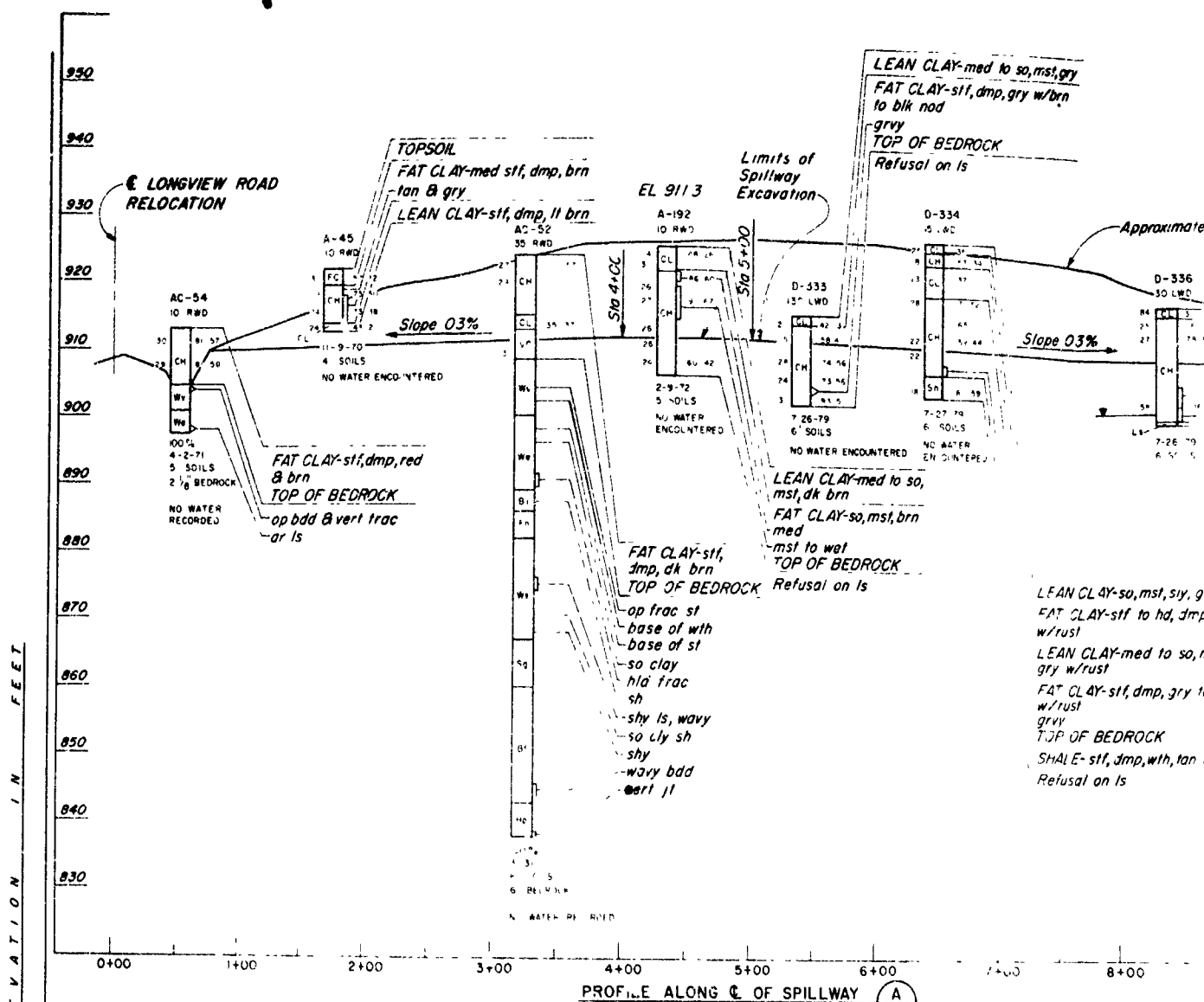
U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY MISSOURI

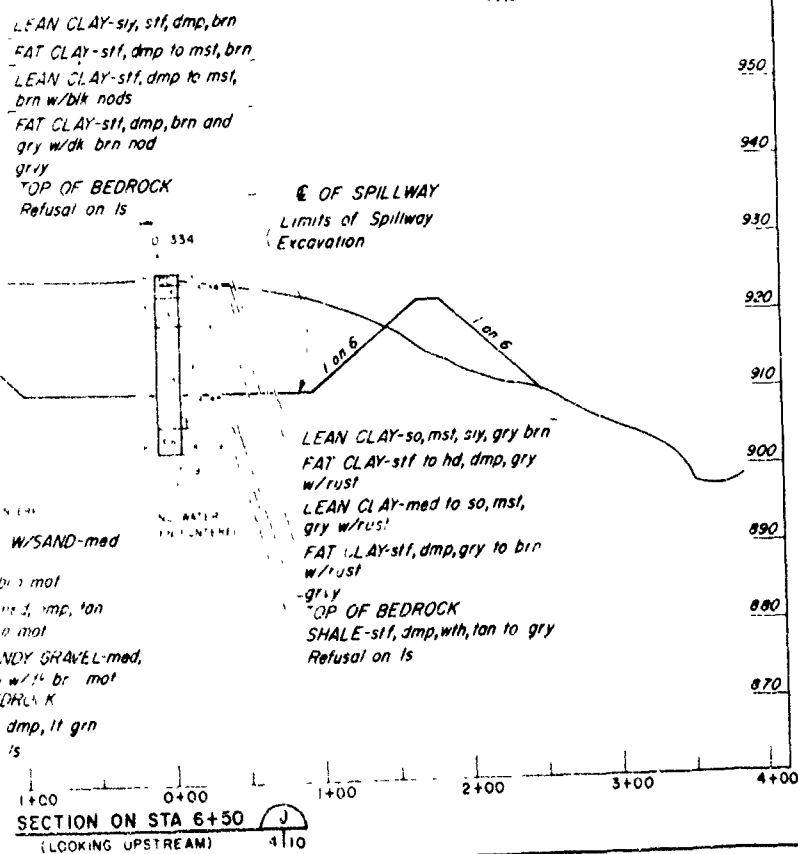
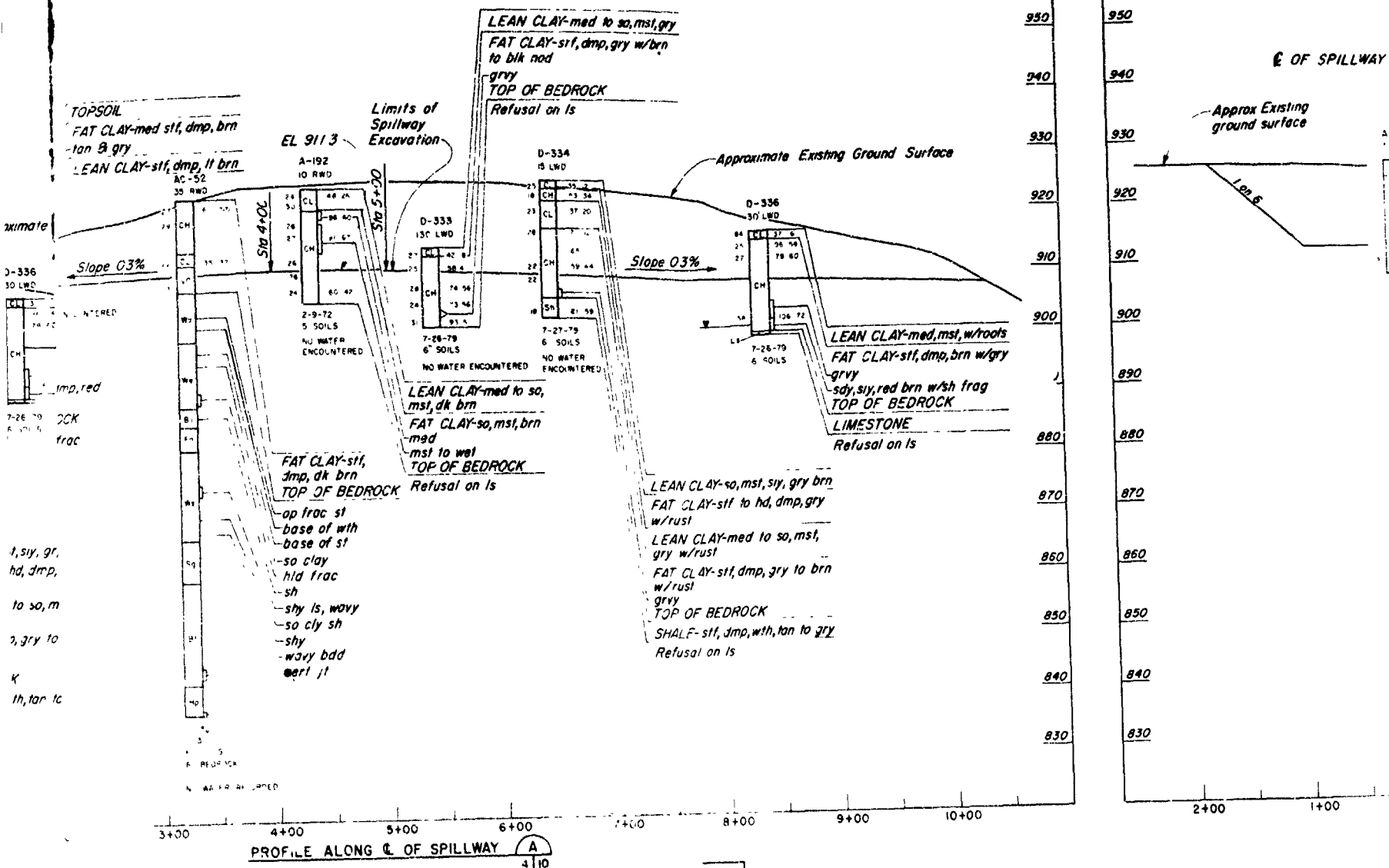
Designed by: R. D.  
Drawn by: R. H.  
Checked by: R. G. D. / W. B. E.  
Date: AS SHOWN

LITTLE BLUE RIVLR, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
OUTLET WORKS, APPROACH, & OUTLET  
CHANNELS GEOLOGIC SECTIONS &  
DETACHED BORINGS

Sheet number: 1  
Date: 10/1/59

RBL-3-1409



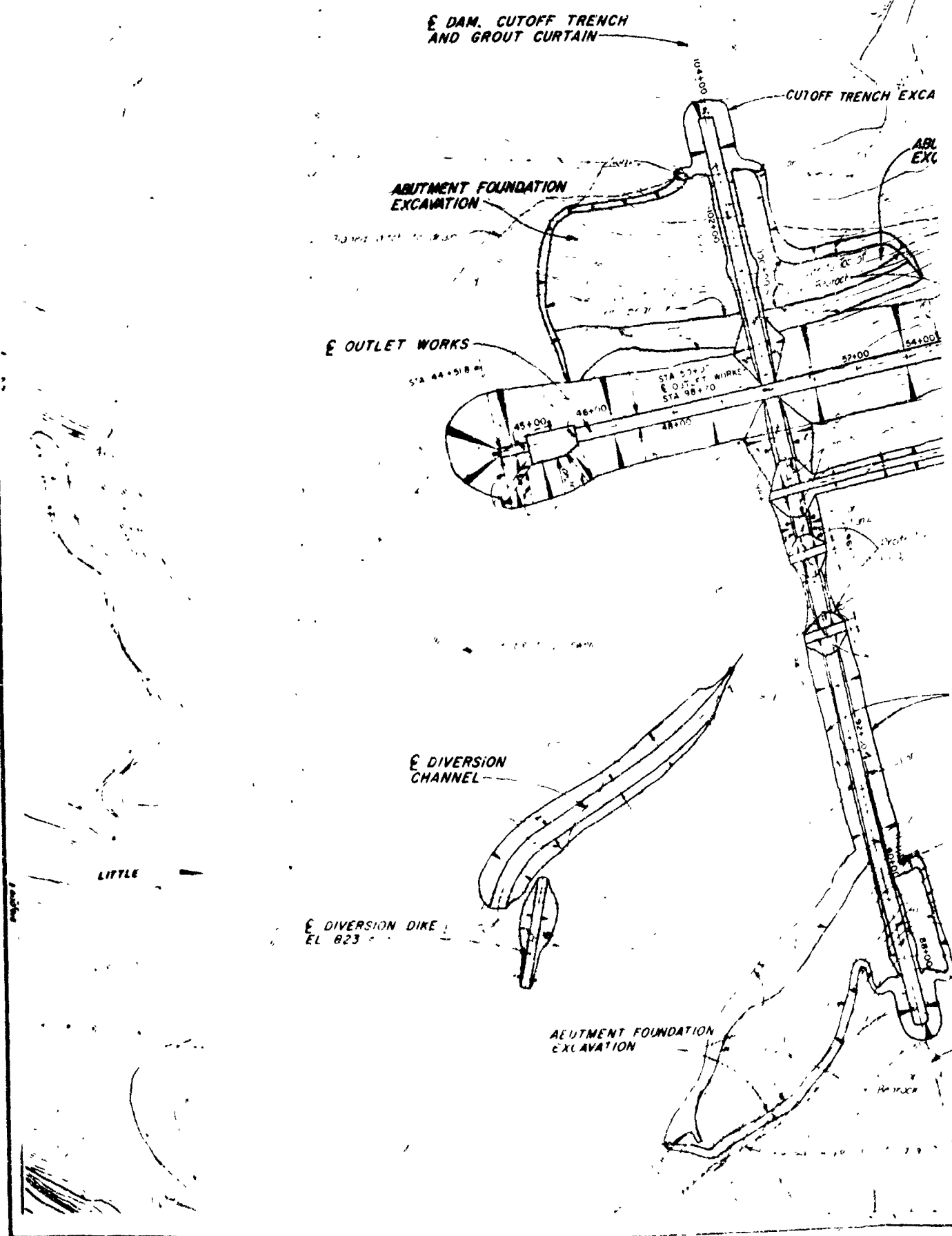


## NOTES

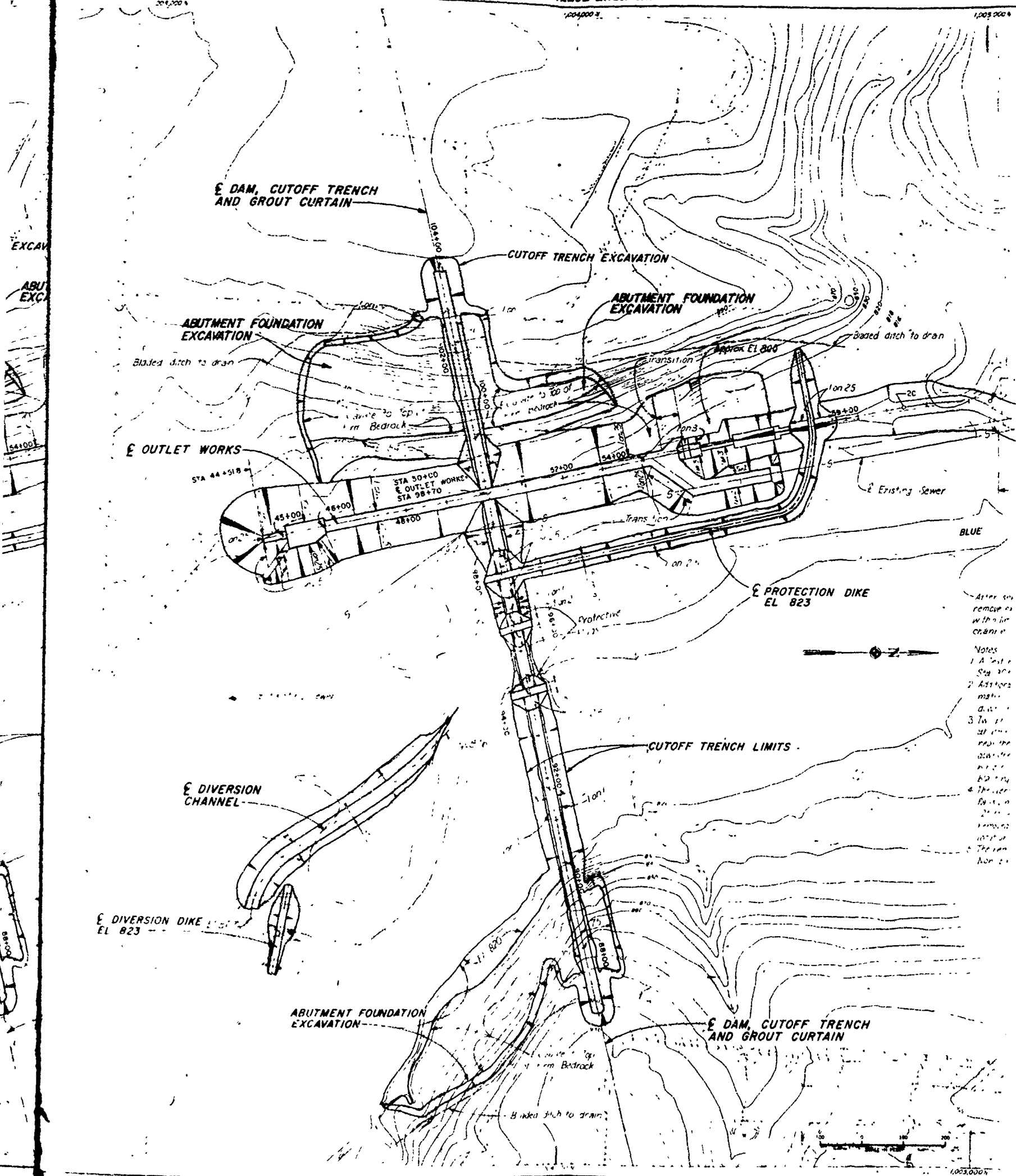
1 Spillway floor El 911.3 between Sta 4+00 and Sta 5+00







LITTLE



DAM, CUTOFF TRENCH  
AND GROUT CURTAIN

CUTOFF TRENCH EXCAVATION

ABUTMENT FOUNDATION  
EXCAVATION

ABUTMENT FOUNDATION  
EXCAVATION

OUTLET WORKS

PROTECTION DIKE  
EL 823

DIVERSION  
CHANNEL

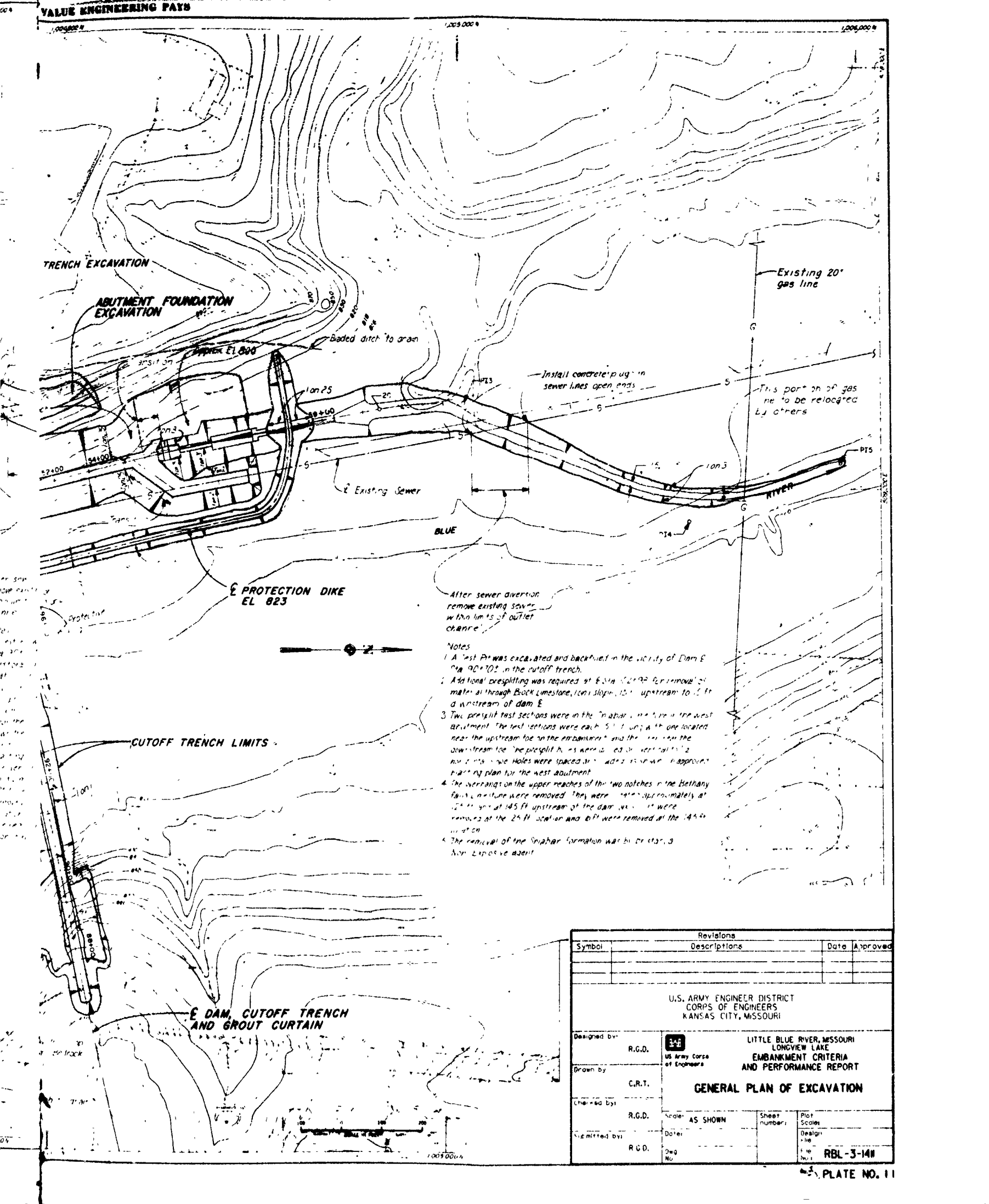
DIVERSION DIKE  
EL 823

ABUTMENT FOUNDATION  
EXCAVATION

DAM, CUTOFF TRENCH  
AND GROUT CURTAIN

- Notes
1. After the removal of the existing structure, the channel shall be widened to 20 feet.
  2. Additions made to the existing structure shall be made in accordance with the plans and specifications.
  3. The existing structure shall be removed in accordance with the plans and specifications.
  4. The existing structure shall be removed in accordance with the plans and specifications.
  5. The existing structure shall be removed in accordance with the plans and specifications.
  6. The existing structure shall be removed in accordance with the plans and specifications.





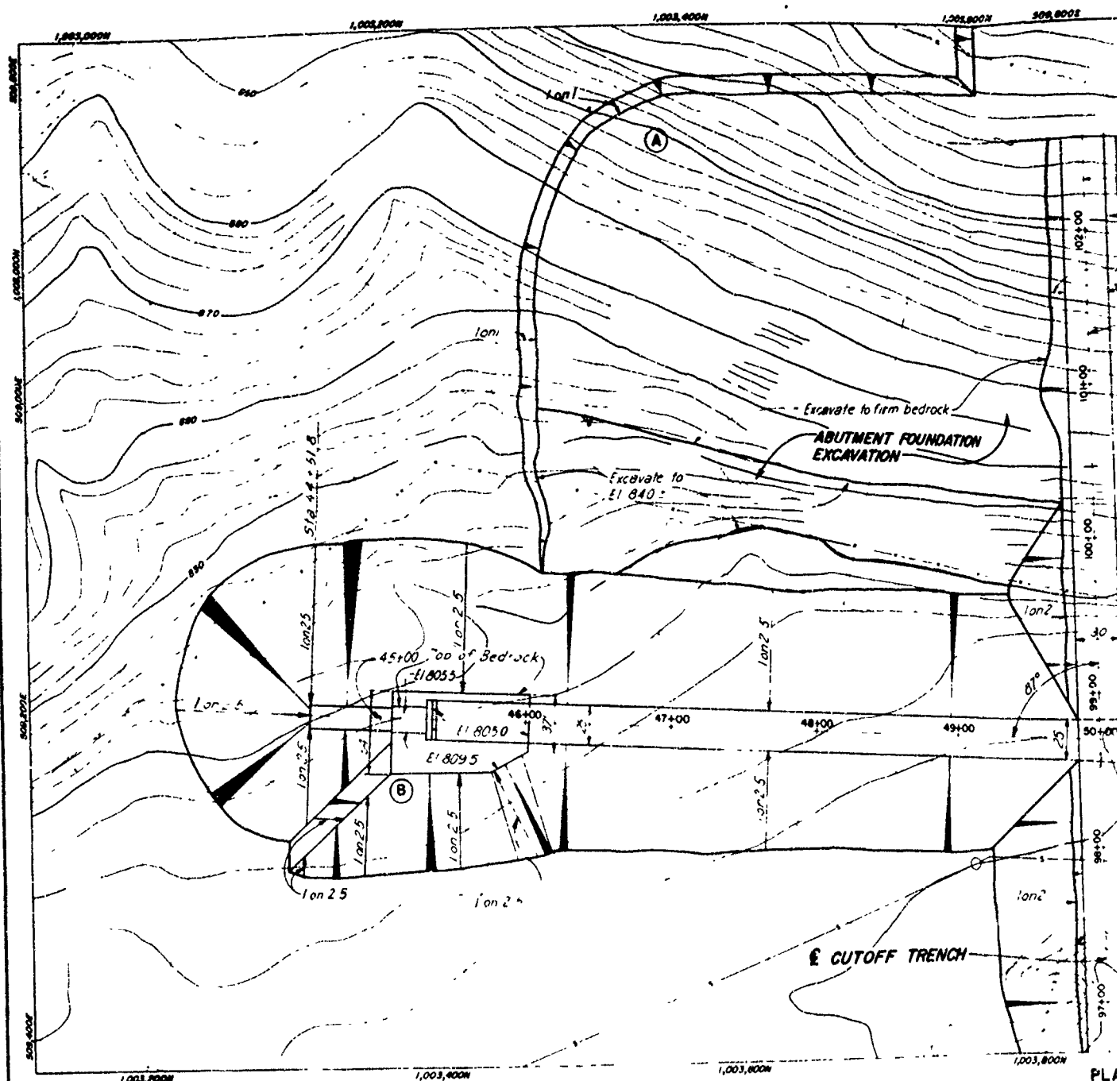
After sewer diversion  
remove existing sewer  
within limits of outlet  
channel

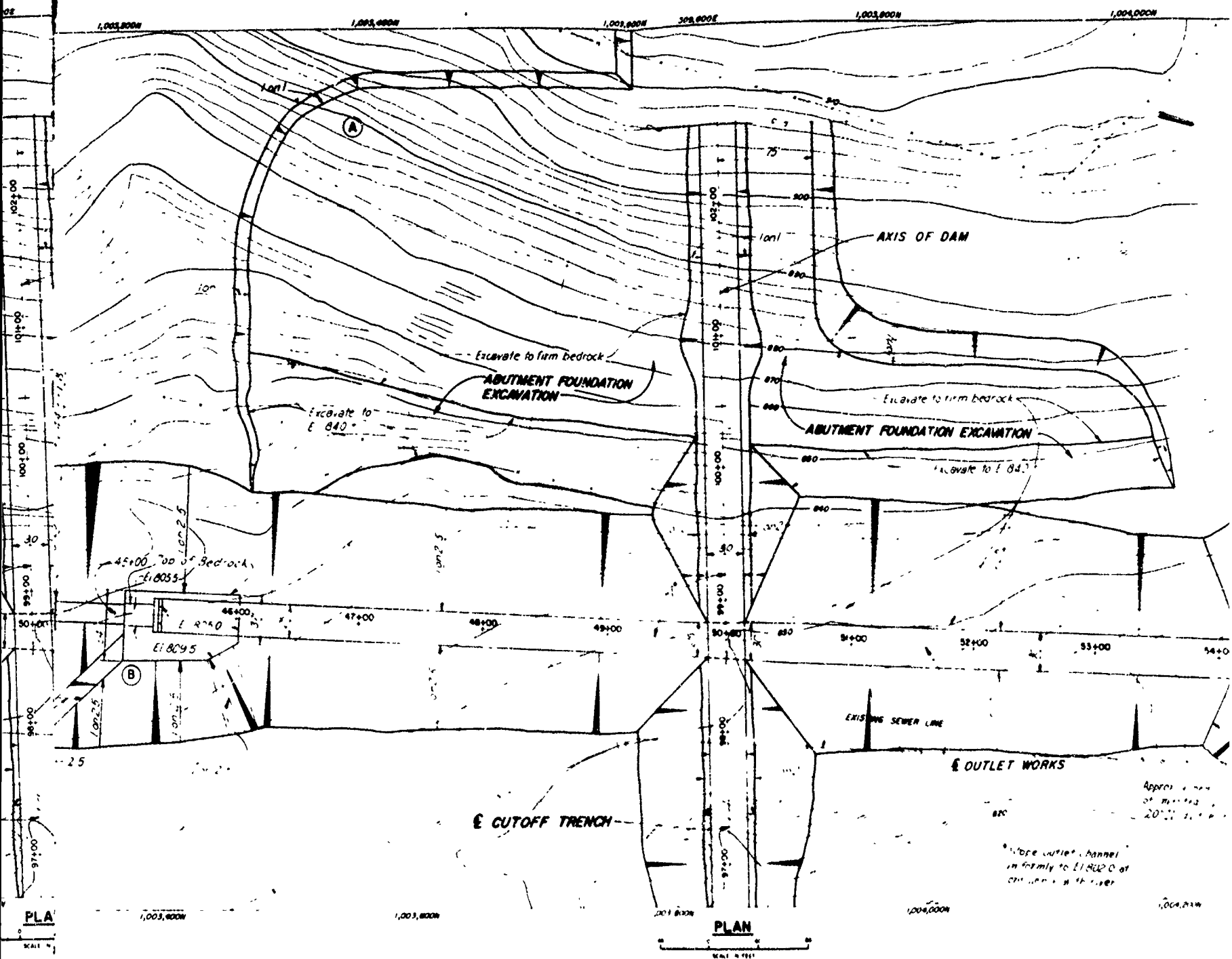
Notes

1. A test pit was excavated and backfilled in the vicinity of Dam E Sta. 90+70.2 in the cutoff trench.
2. Additional presplitting was required at E Sta. 12+92 for removal of material through Block Limestone, 10m slope, 12 ft upstream to 12 ft downstream of dam E.
3. Two presplit test sections were in the Snaphar formation in the west abutment. The test sections were each 5 ft long with one located near the upstream toe on the embankment and the other near the downstream toe. The presplit holes were spaced at 12 ft vertically and horizontally. Holes were spaced at 12 ft upstream in appropriate planning plan for the west abutment.
4. The overhangs on the upper reaches of the two notches in the Bethany Bank Limestone were removed. They were removed approximately at 12 ft from the 145 ft upstream of the dam. The 12 ft were removed at the 24 ft location and 6 ft were removed at the 145 ft location.
5. The removal of the Snaphar formation was by blasting. Non-explosive agent.

Revisions			
Symbol	Descriptions	Date	Approved

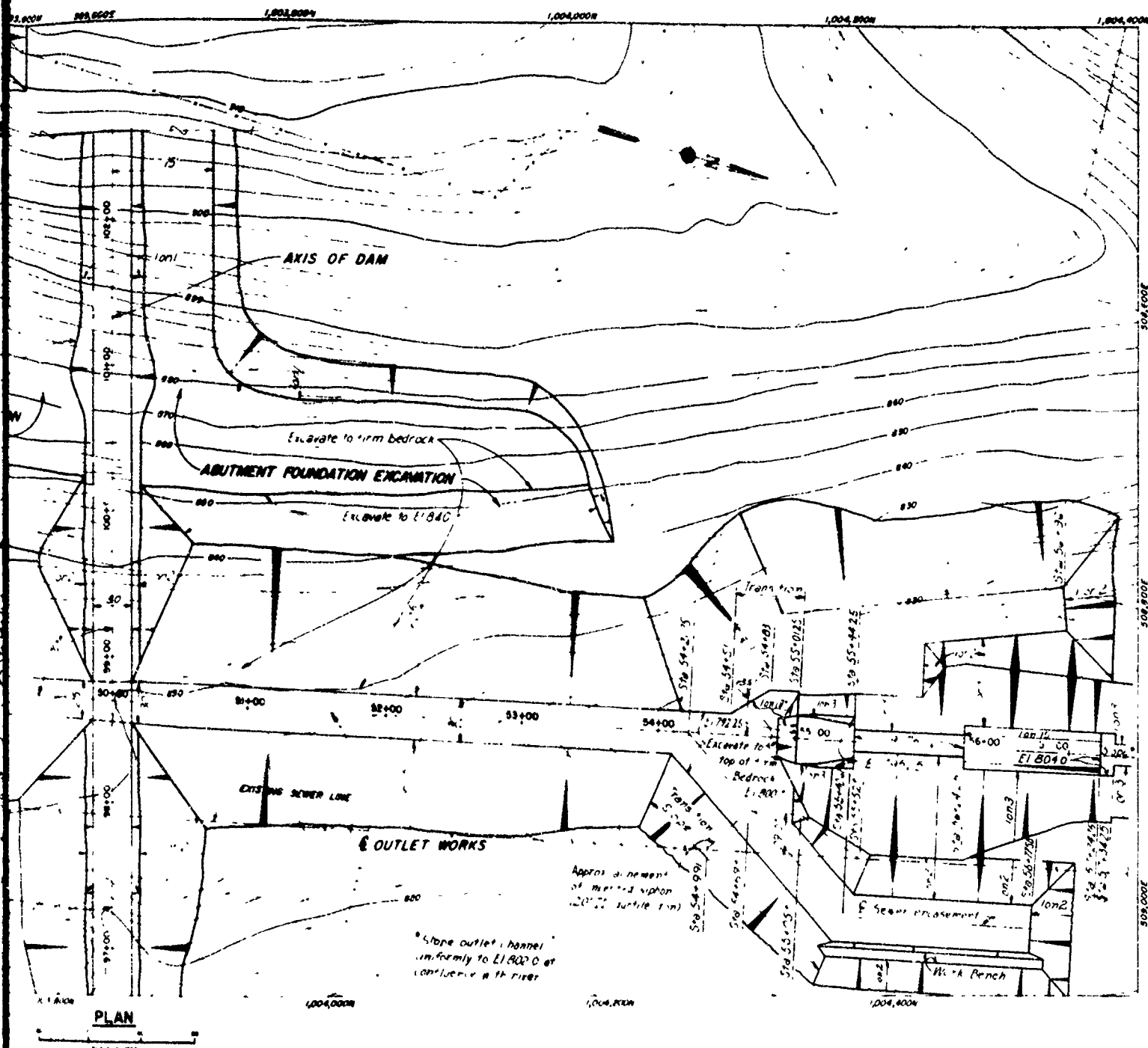
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by: R.G.D.	346 U.S. Army Corps of Engineers	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by: C.R.T.		GENERAL PLAN OF EXCAVATION	
Checked by: R.G.D.	Scale: AS SHOWN	Sheet number: 	Plot Scale: 
Submitted by: R.G.D.	Date: 	Design File: 	
	Dwg No: 		RBL-3-14H





YG'FS

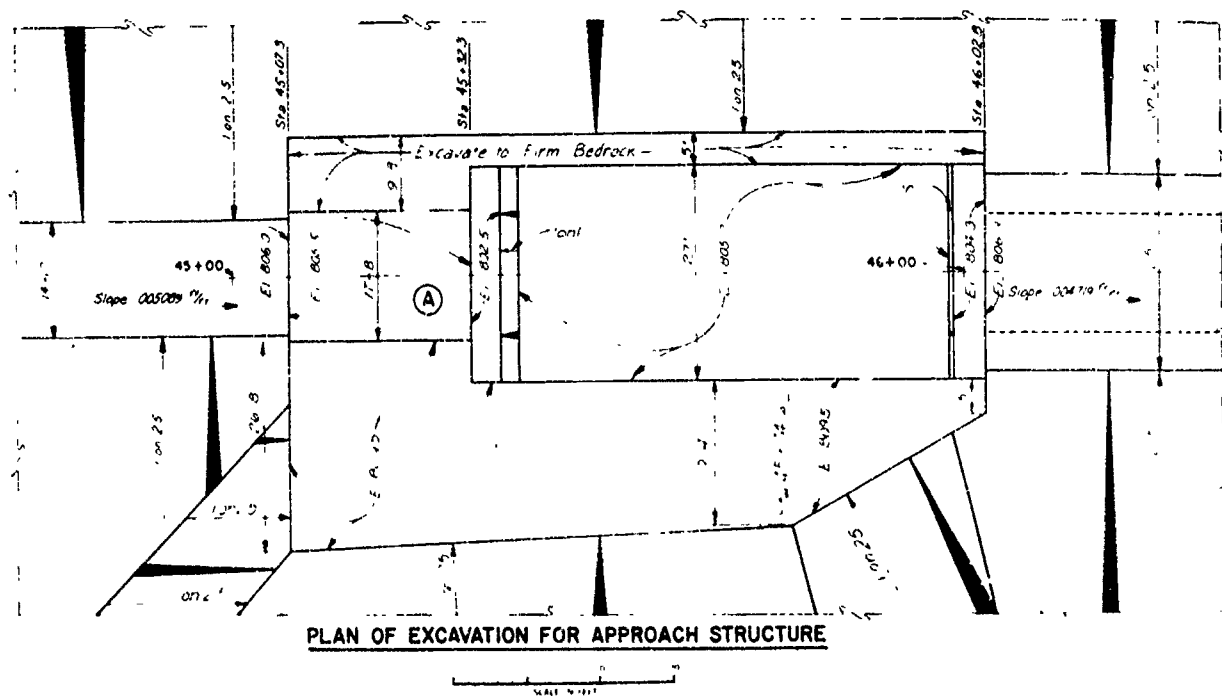
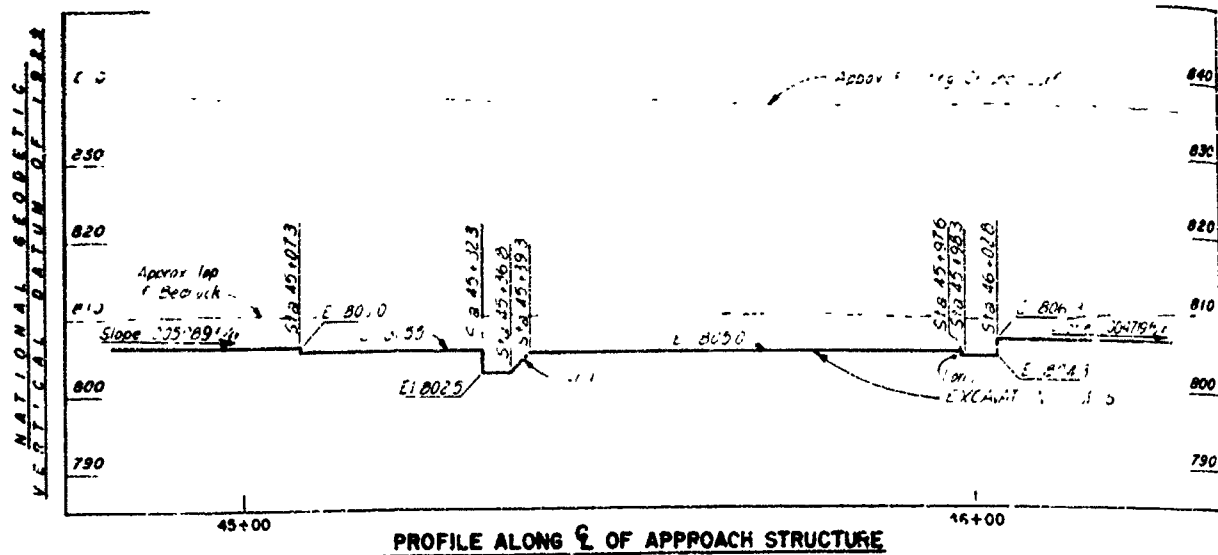
Additional work required in the  
nature of relief works \$52,400, less  
Part 1 remove the unsold material  
\$1,000 = 1 yds of slide material removed  
\$1,000 = yds of additional material  
to be  
to be  
to be



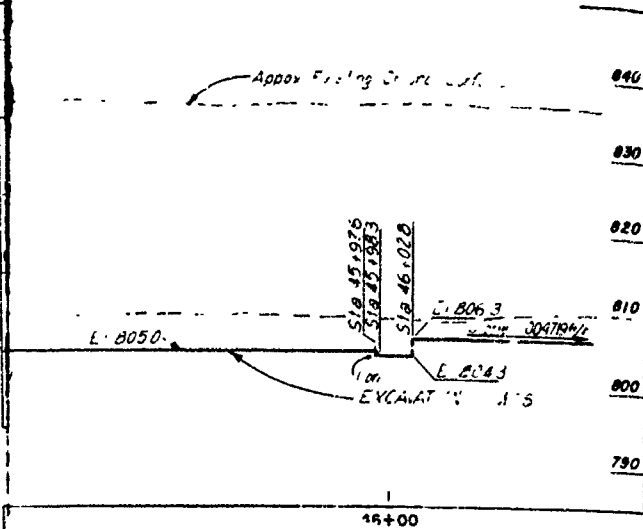
#### Notes

1. Additional excavation was required in the vicinity of Outlet works Sta 52+00, left Bank, to remove the land slide material at
  - A 20,000 cu yds of slide material removed
  - B 20,000 cu yds of additional impervious backfill
  - C Additional clearing and grubbing required

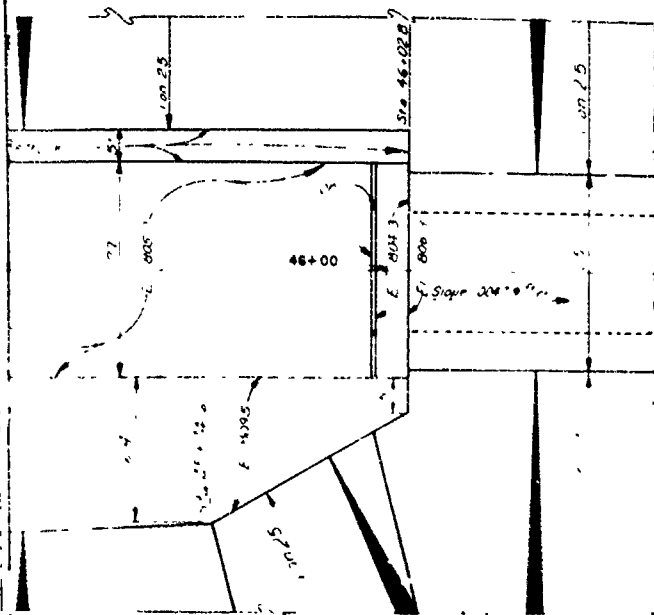
Revisions			
Symbol	Descriptions	Date	Approved
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by: R.G.D. Drawn by: J.E.R. Checked by: R.G.D. Entered by: R.G.D.		LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT  <b>OUTLET WORKS            PLAN OF EXCAVATION</b>	
Scale:	AS SHOWN	Sheet Number:	Not Scored
Date:			Design File:
RBL-3-1412			The No.



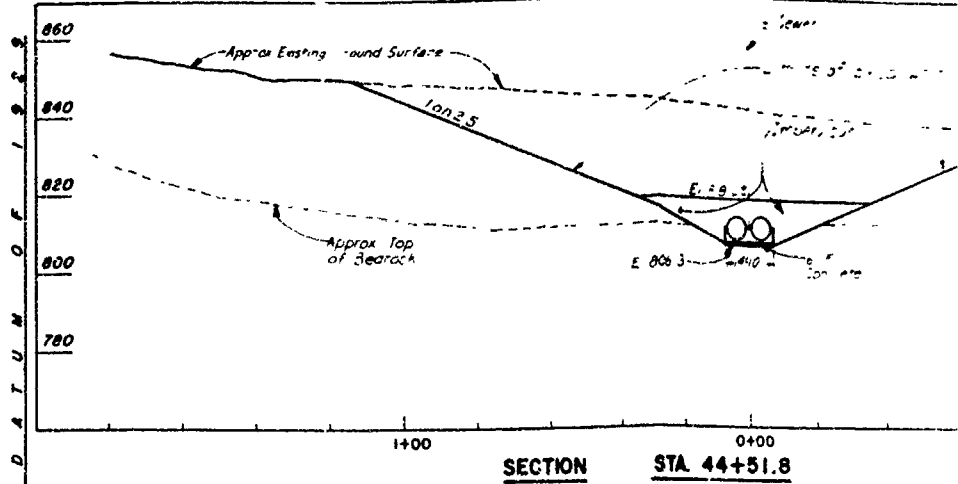
V A - I - A - S I - O - E - T - I - C V E R T - I - C A L D A T U M O F 1 9 2 9



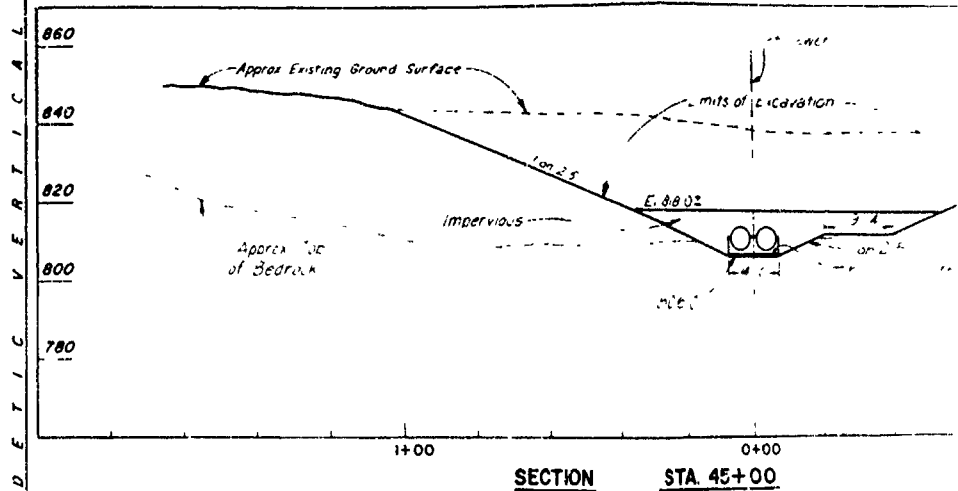
OF APPROACH STRUCTURE



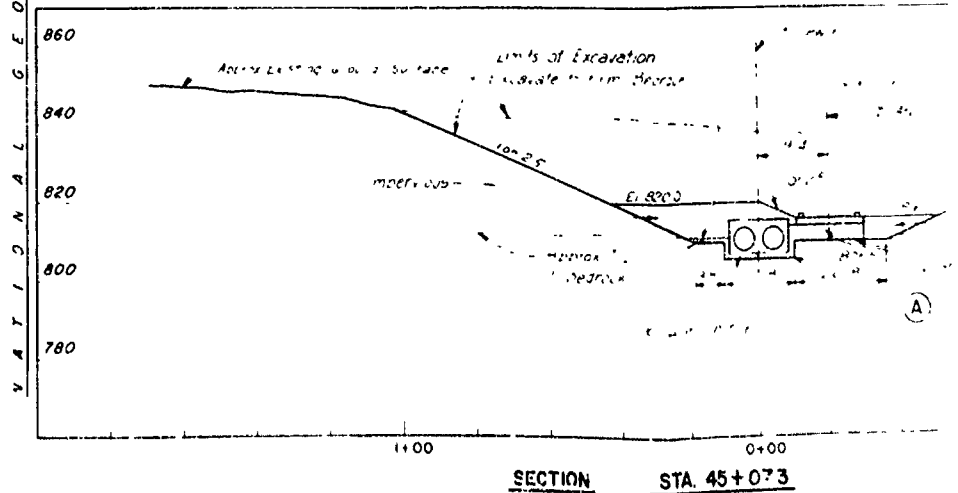
APPROACH STRUCTURE



SECTION STA. 44+51.8

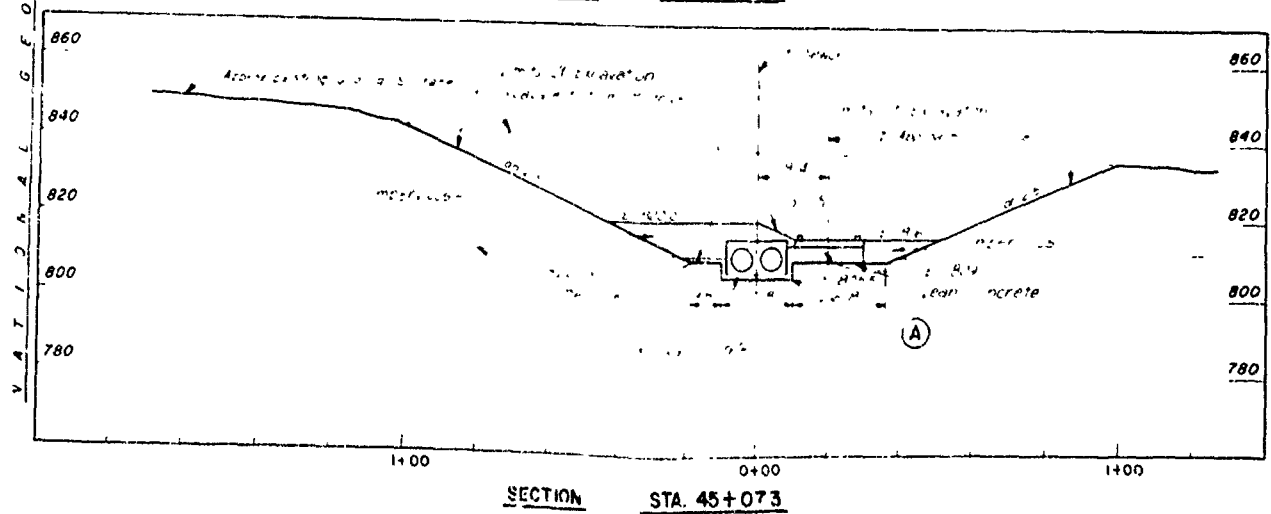
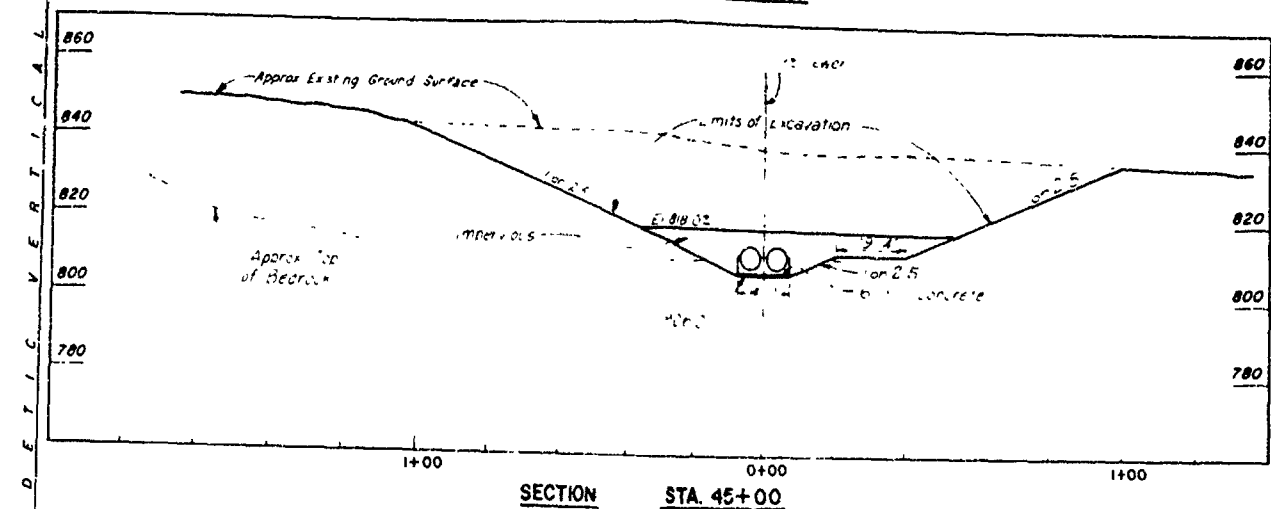
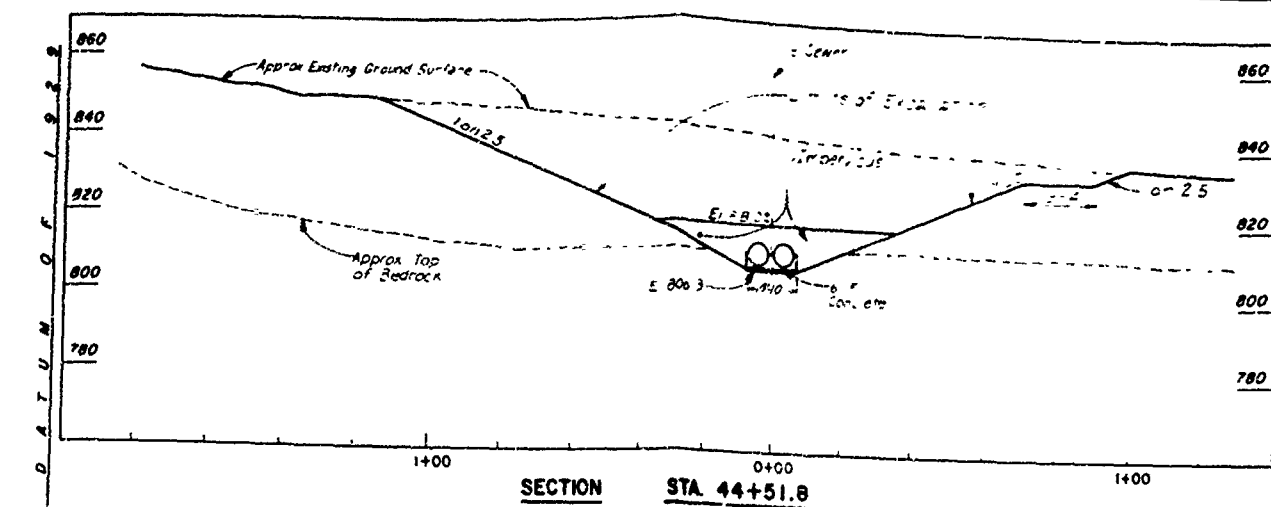


SECTION STA. 45+00



SECTION STA. 45+07.3

Design	
Check	
Approved	
Remarks	



Symbol	Revisions Descriptions	Date	Approved

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: R.G.D.

Drawn by: J.G.S.

Checked by: R.G.D.

Approved by: R.G.D.

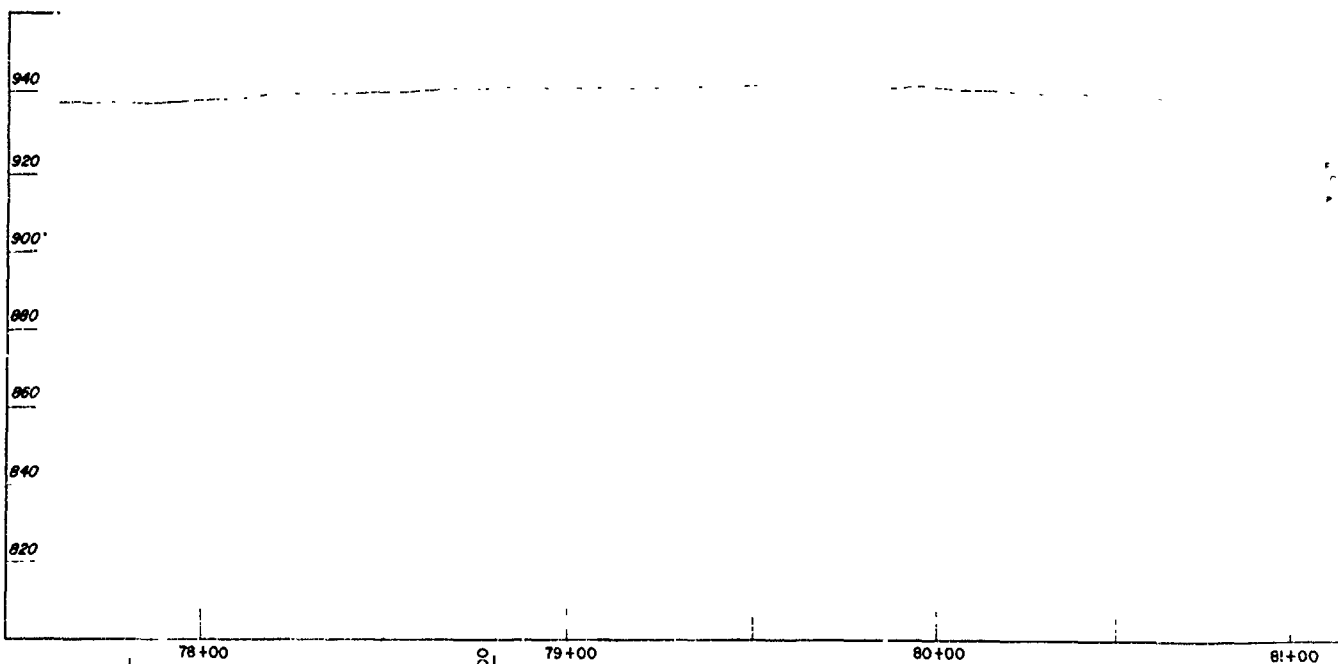
LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

APPROACH STRUCTURES EXCAVATION  
PLAN, PROFILE, & SECTIONS

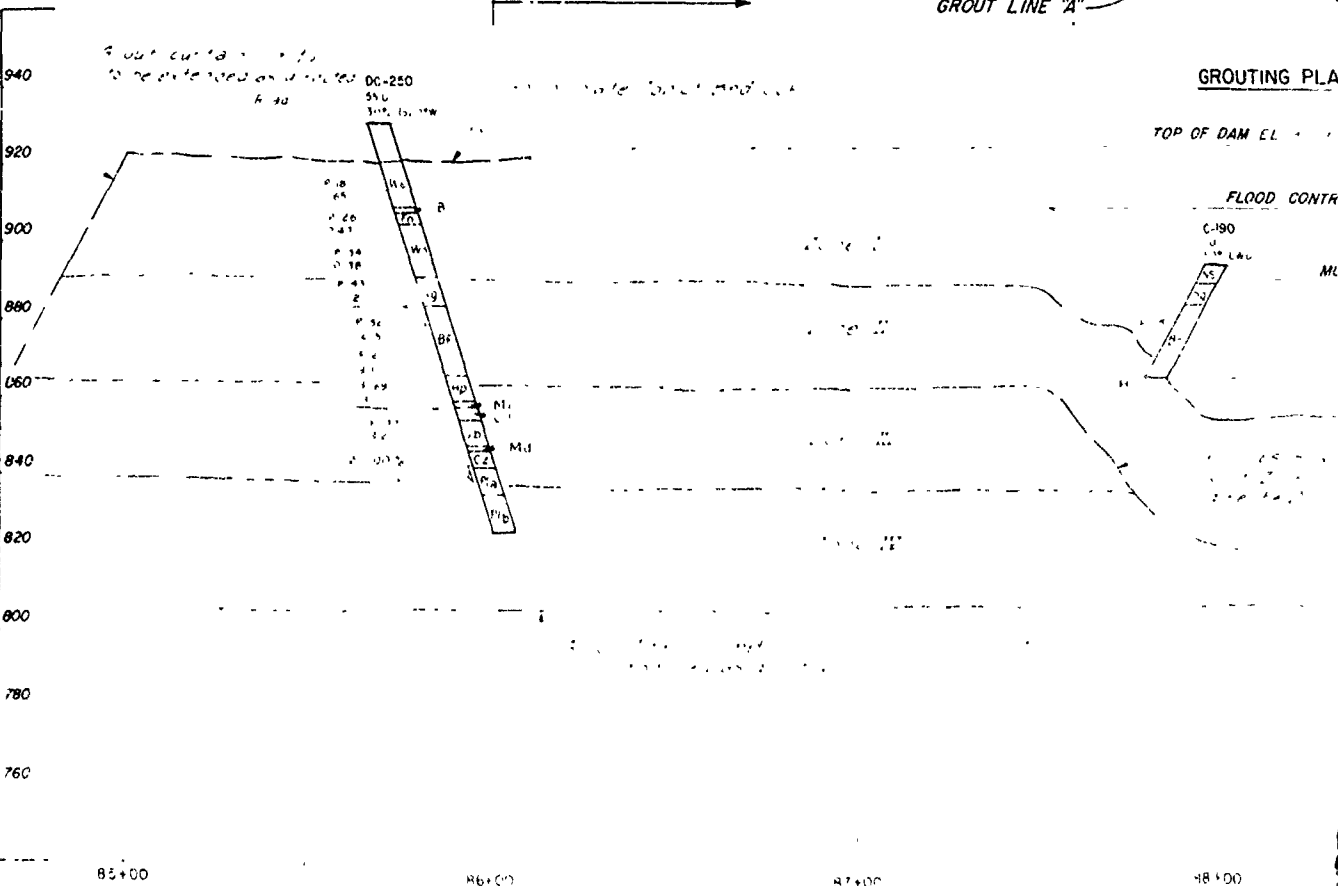
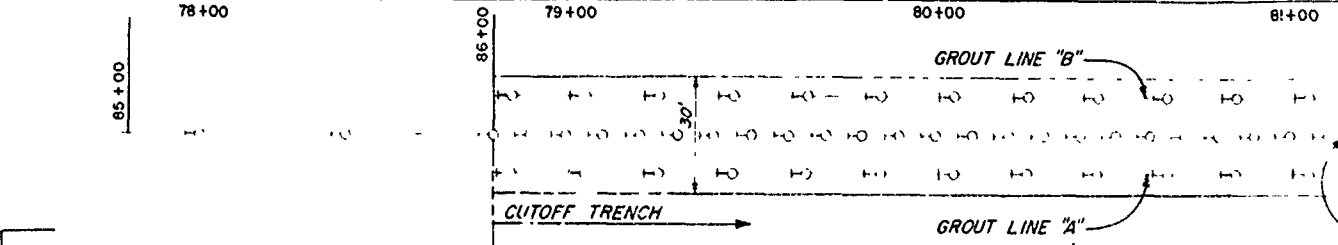
AS SHOWN

Sheet Number: 1 of 1

Project Number: RBL-3-1413



# ANNUAL GEODETIC VERTICAL DATUM OF 1929



## GROUTING PLAN

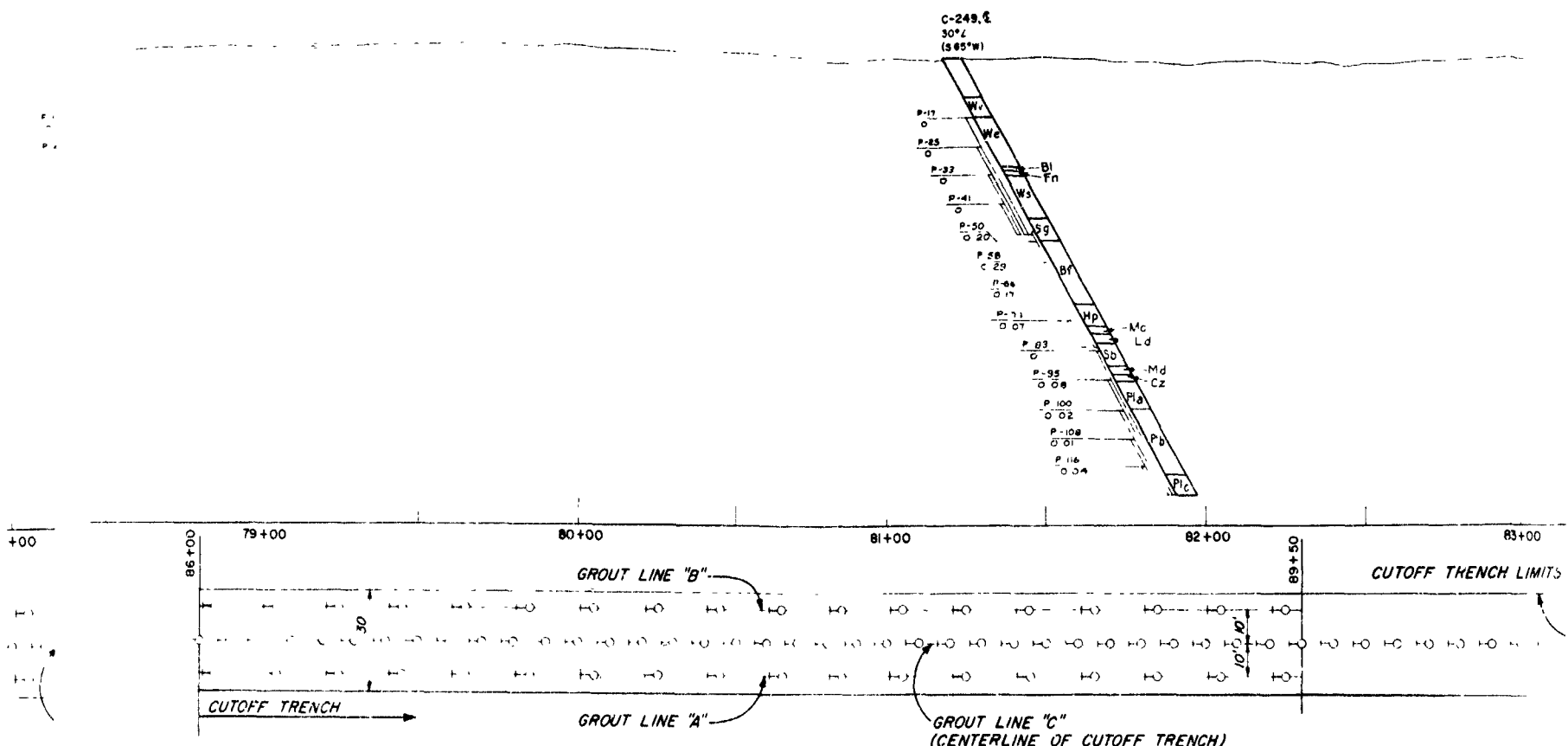
TOP OF DAM EL. 4.1

FLOOD CONTR

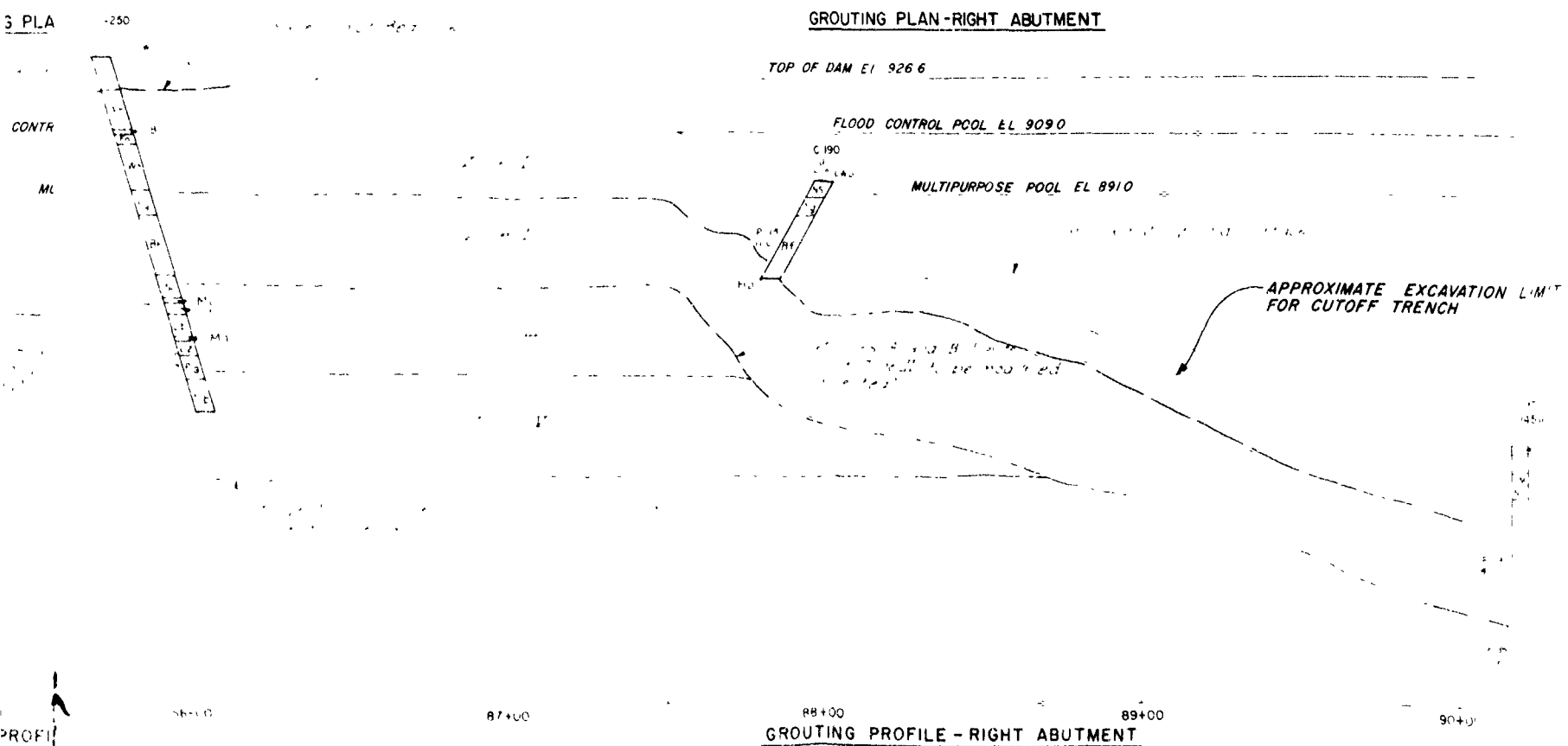
ML

10-500  
GROUTING PROFIT





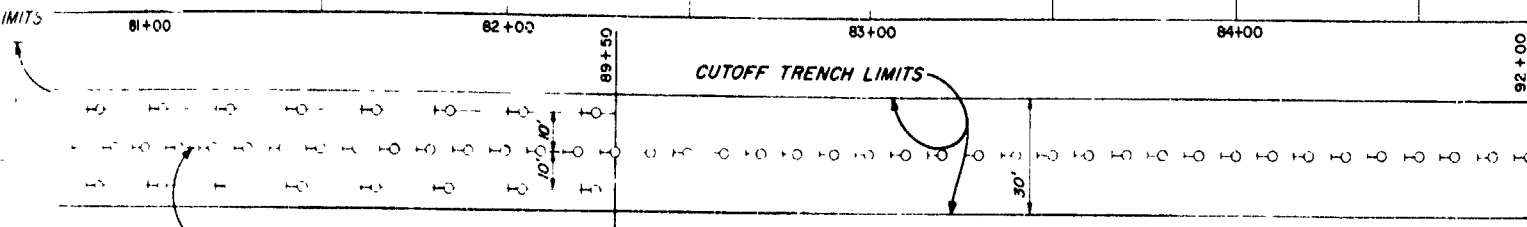
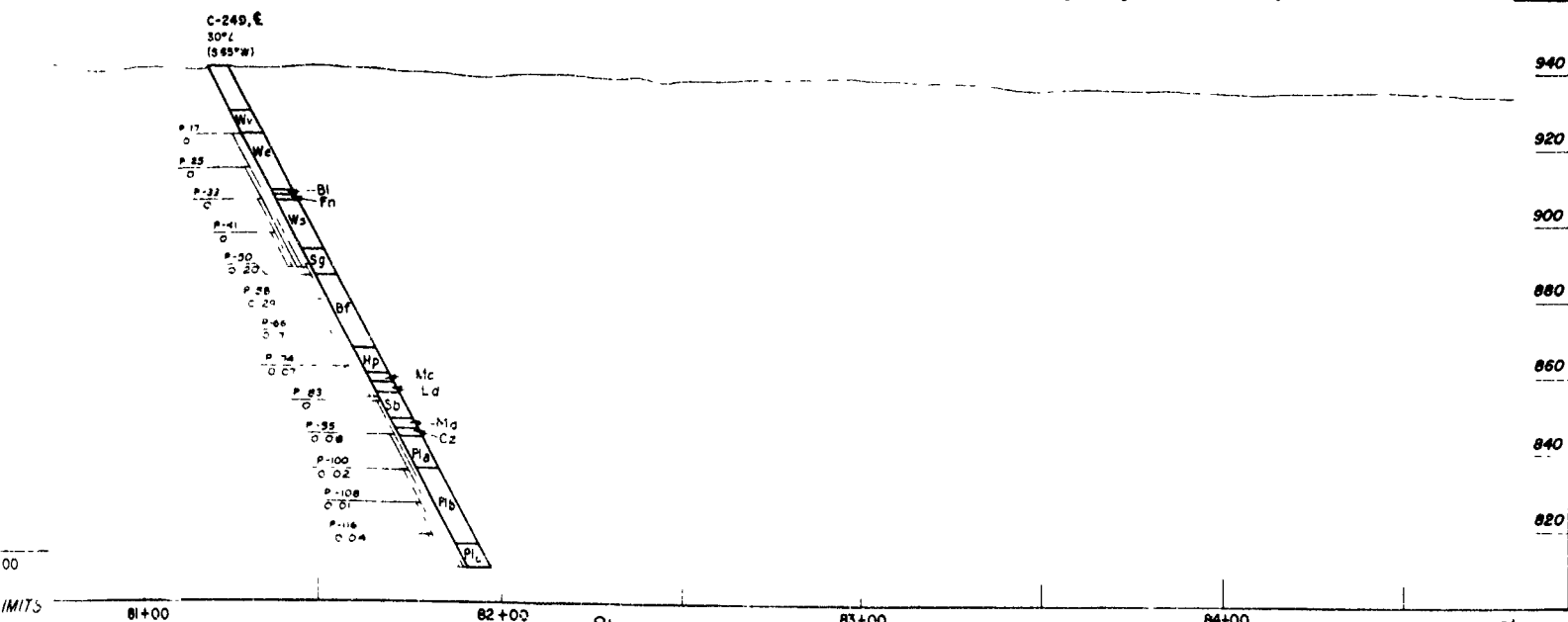
GRROUTING PLAN-RIGHT ABUTMENT



GRROUTING PROFILE - RIGHT ABUTMENT

NOTES

For plan of Borings, See Plate No. 3  
 For General Geologic Column and Legend, See Plate No. 6  
 For boring logs showing soil and bedrock description, See Plate No. 7  
 Only primary holes are shown on plans. Within the limits of the cutoff trench, secondary grout holes shall be mandatory  
 Indicates direction of grout holes (Direction may be changed by the Contracting Officer)



GROUTING PLAN-RIGHT ABUTMENT

OF DAM EL 926.6


FLOOD CONTROL POOL EL 909.0

MULTIPURPOSE POOL EL 891.0

APPROXIMATE EXCAVATION LIMITS FOR CUTOFF TRENCH

Revisions		Date	Approved
Number	Description		

U.S. ARMY ENGINEER DISTRICT  
 CORPS OF ENGINEERS  
 KANSAS CITY, MISSOURI

Design by: R.G.D.  Little Blue River, Missouri  
 LONGVIEW LAKE  
 EMBANKMENT CRITERIA  
 AND PERFORMANCE REPORT

Checked by: J.G.S.

Drawn by: R.G.D. AS SHOWN

Reviewed by: R.G.D.

Project Number: RBL-3-1414



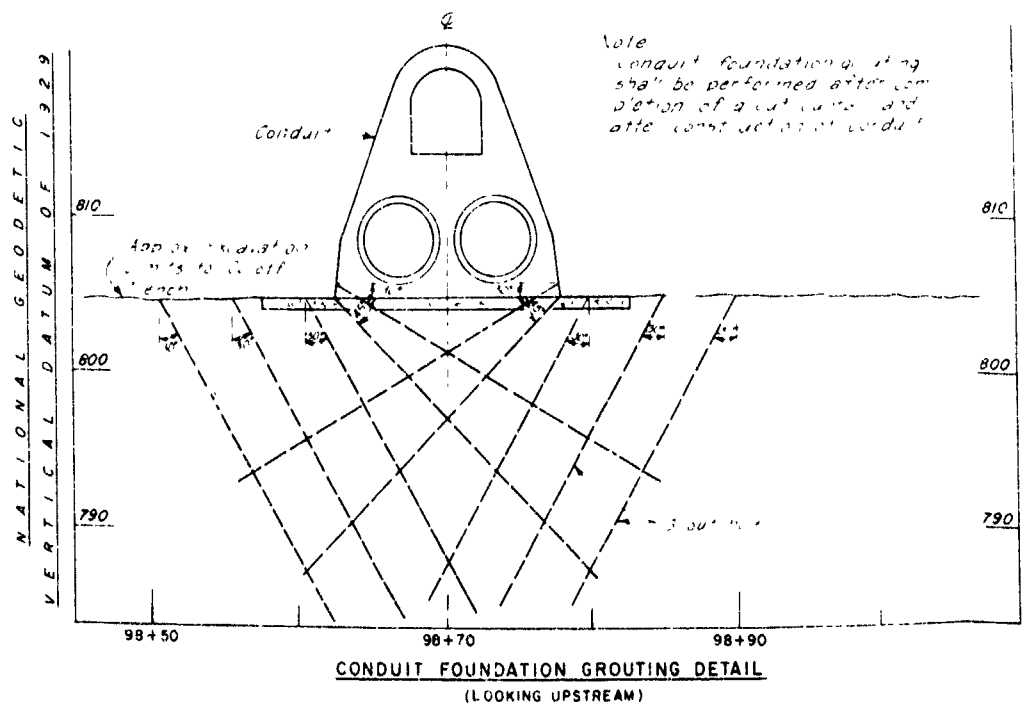
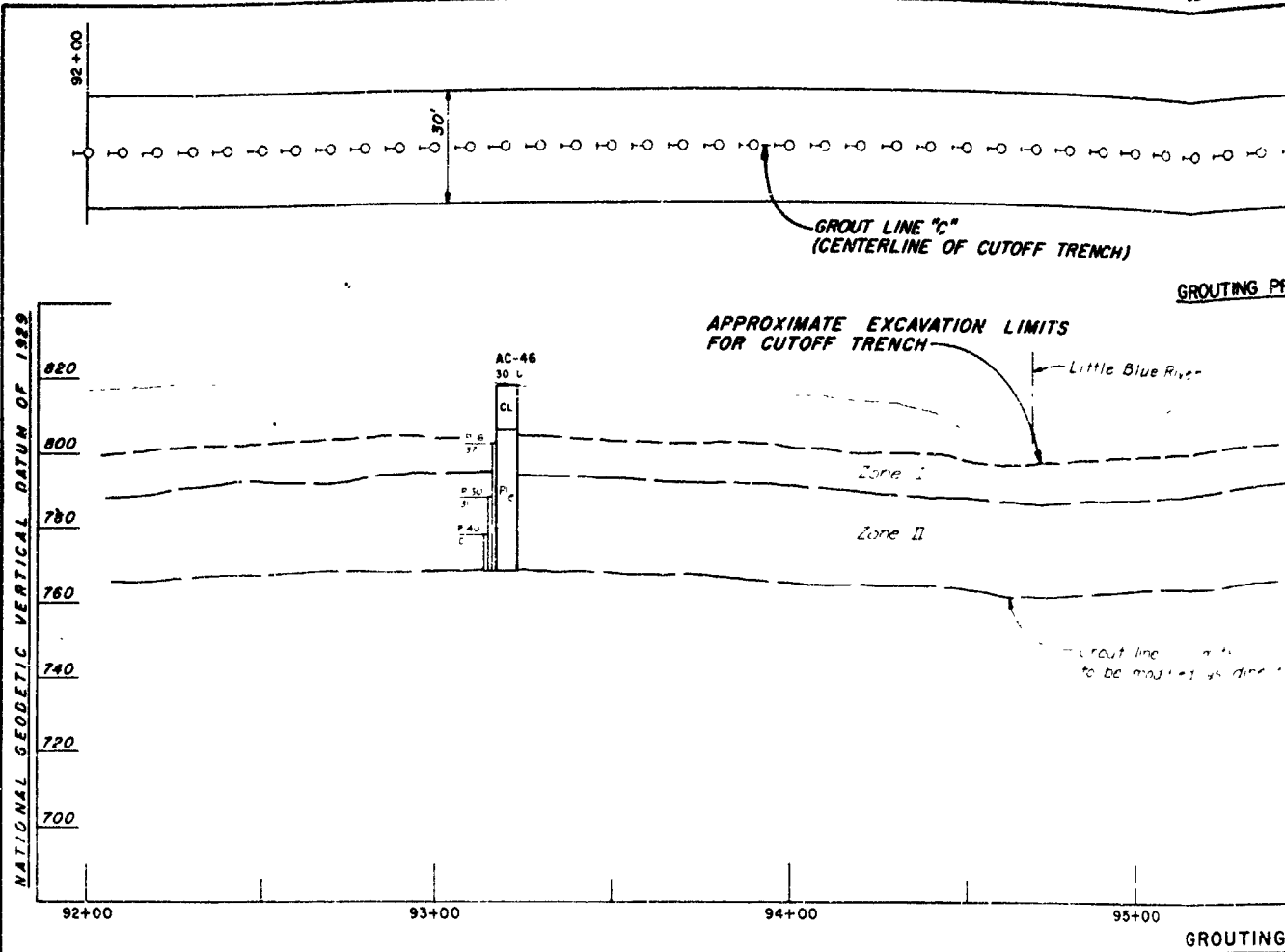
GROUTING PROFILE - RIGHT ABUTMENT

69+00

70+00

71+00

72+00



ENGINE

ING PRO

FING

CUTOFF TRENCH LIMITS

GROUT LINE "C"  
(CENTERLINE OF CUTOFF TRENCH)

GROUTING PROFILE - VALLEY

APPROXIMATE EXCAVATION LIMITS  
FOR CUTOFF TRENCH

Little Blue River

Zone I

Zone II

UC-9A

UC-9

25 D

CL

PC

LDW 100%

LDW 50%

LDW 25%

P. 30

LDW 100%

PC

GROUT LINE LIMITS  
TO BE MODIFIED AS DIRECTED

93+00

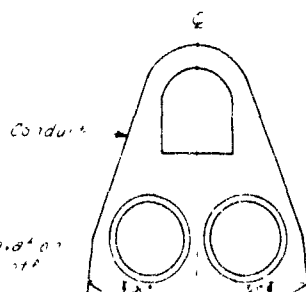
94+00

95+00

96+00

97+00

GROUTING PROFILE - VALLEY



Note:  
Conduit foundation grouting  
shall be performed after com-  
pletion of a cut and haul  
after construction of conduit.

810

800

790

98+50

98+70

98+90

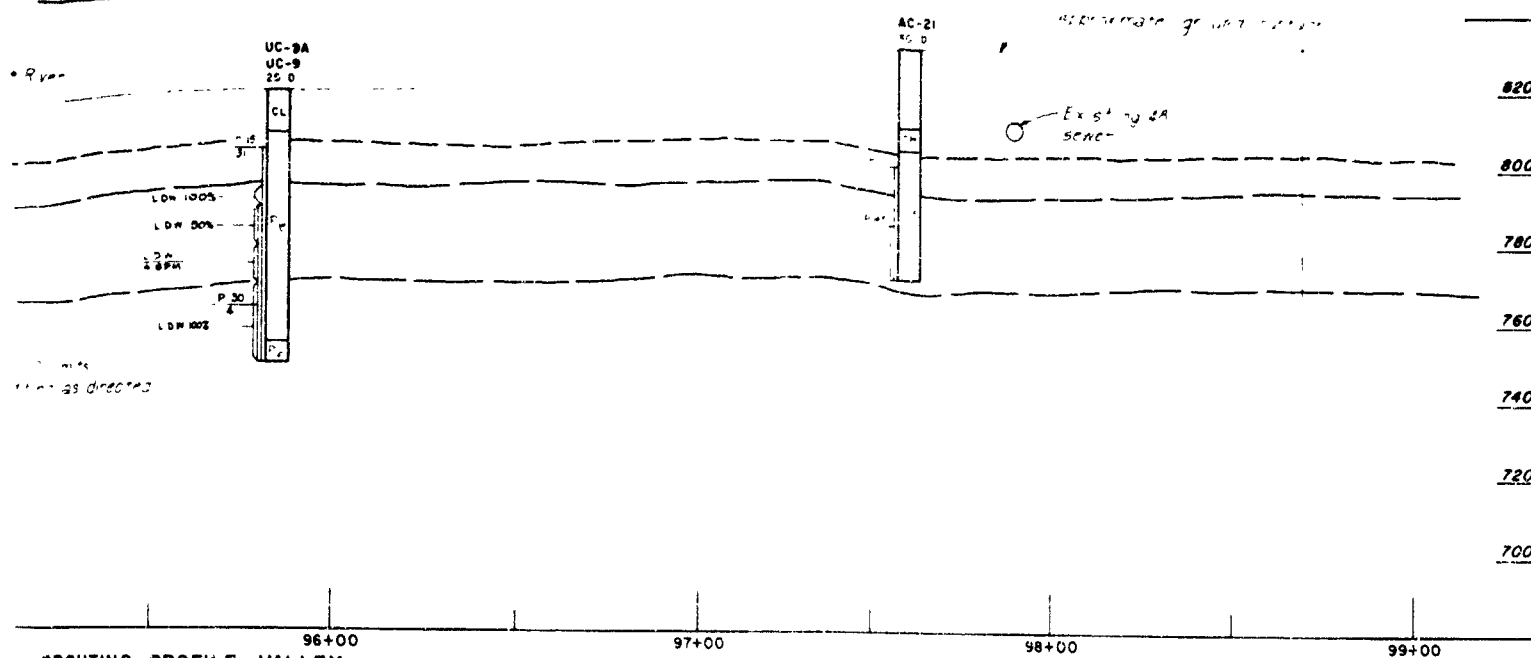
CONDUIT FOUNDATION GROUTING DETAIL

(LOOKING UPSTREAM)

CUTOFF TRENCH LIMITS

Outlet Works

GROUTING PROFILE - VALLEY



GROUTING PROFILE - VALLEY

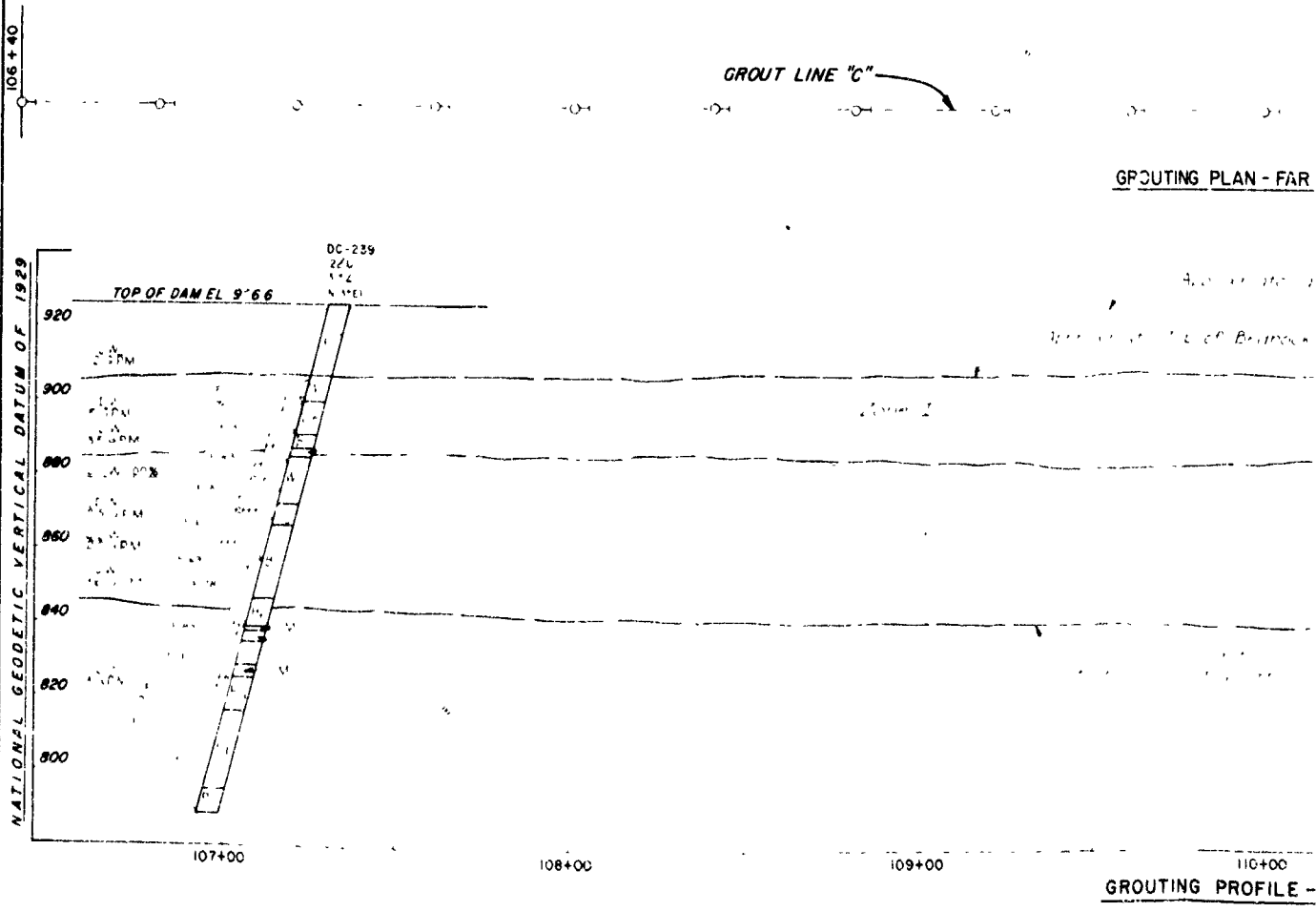
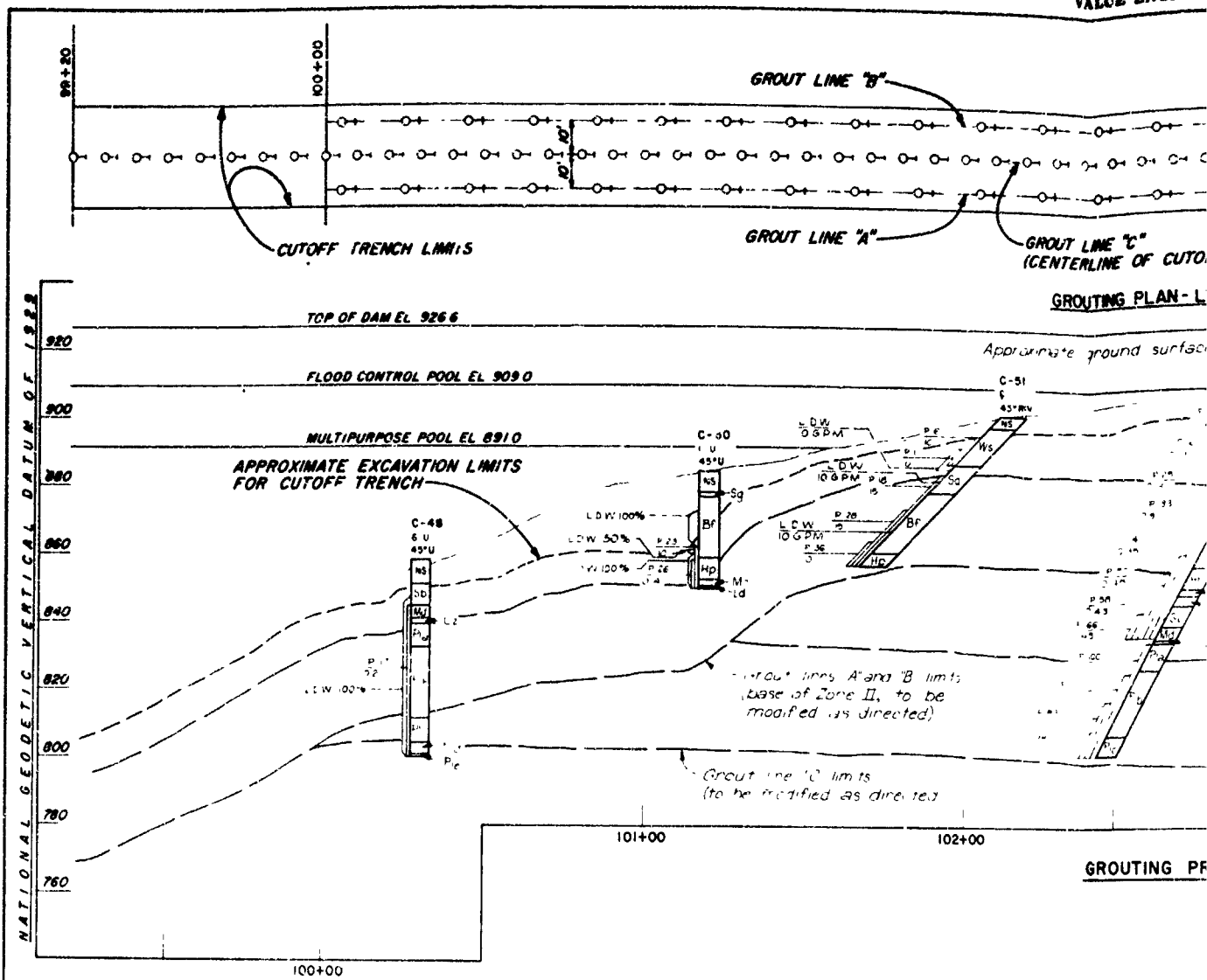
For grout hole logs, see Plate No. 3  
 For grout hole logs, see Plate No. 4  
 For grout hole logs showing soil and bedrock description  
 see Plate No. 5  
 Only primary holes are shown on plans. Within the  
 limits of the cutoff trench, secondary grout holes shall  
 be mandatory.  
 • indicates direction of grout holes. (Direction may be  
 changed by the Contracting Officer)

FOR THE  
 U.S. ARMY  
 ENGINEERS

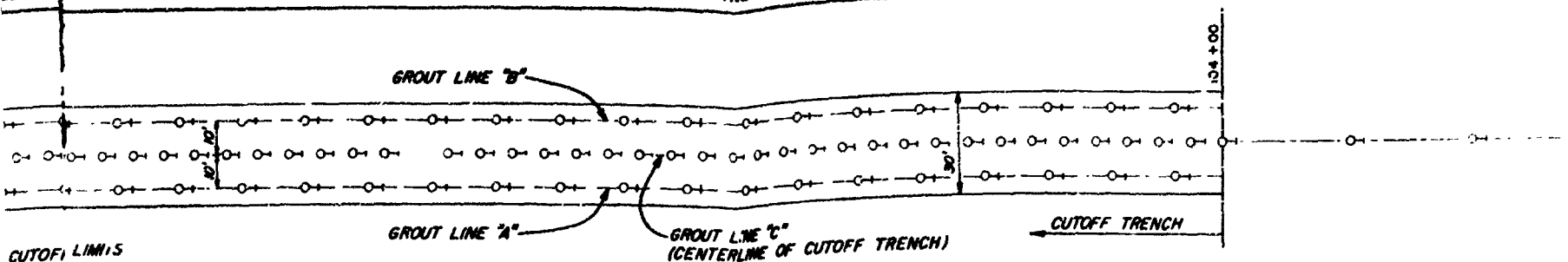
LITTLE BLUE RIVER, MISSOURI  
 LINDVIEW LAKE  
 EMBANKMENT CRITERIA  
 AND PERFORMANCE REPORT

CURTAIN GROUTING PLAN AND PROFILE  
 VALLEY AND CONDUIT FOUNDATION

RBL-3-1415

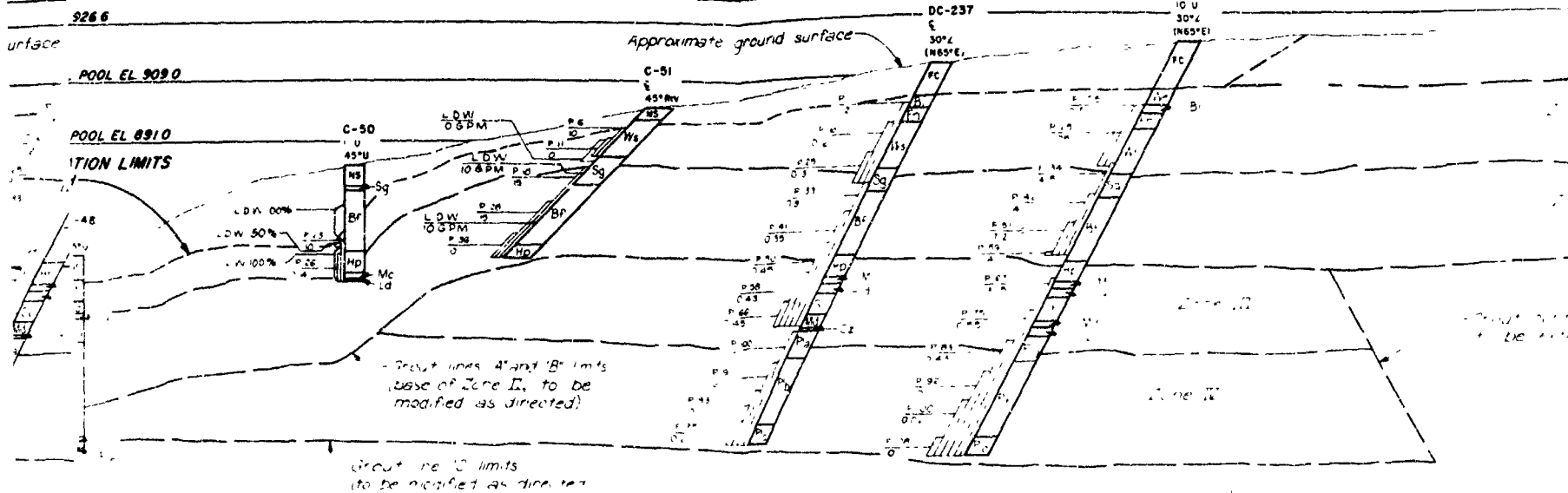


CINER



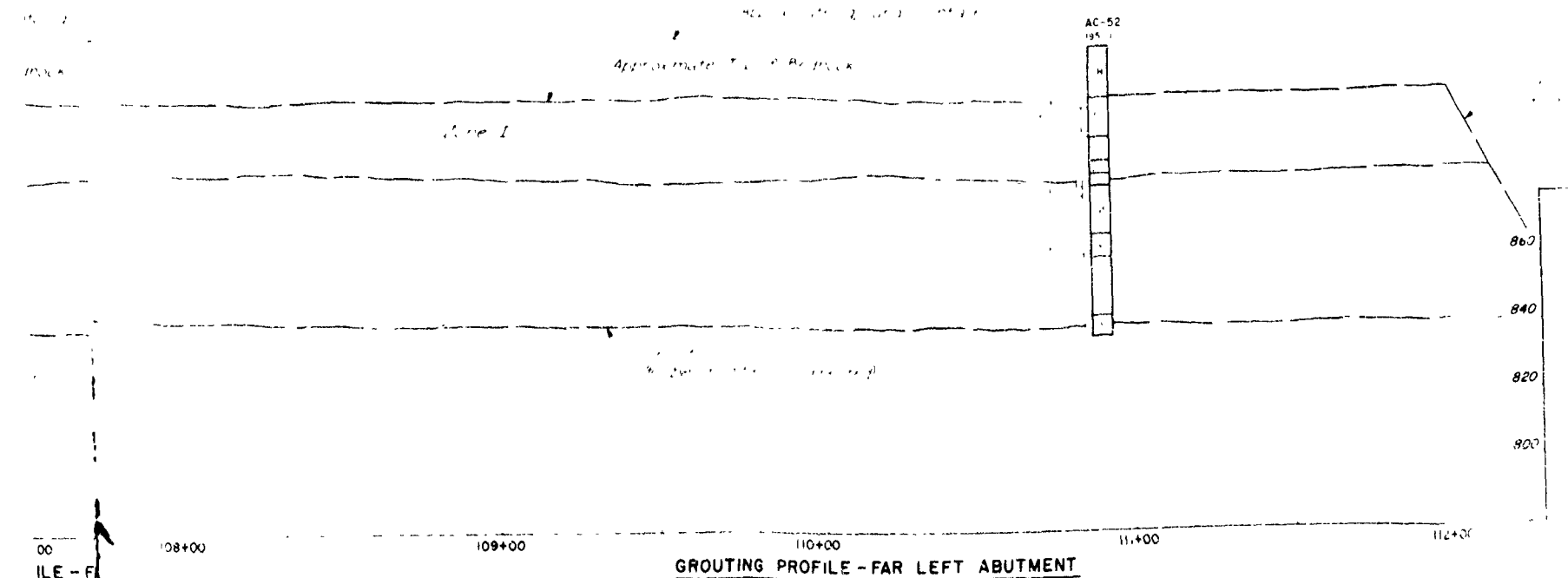
CUTOFF LIMITS

AN - LEI



G PRC

- FAR L



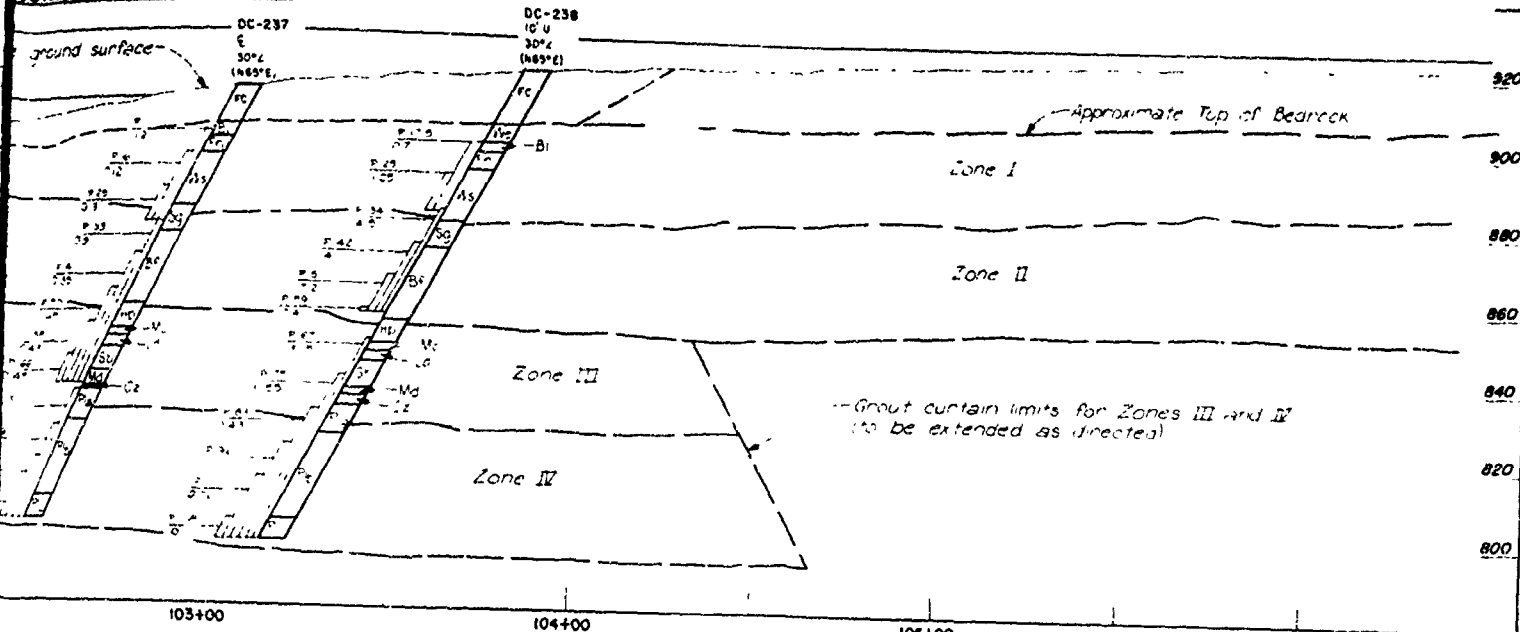
ILE - F

GROUTING PROFILE - FAR LEFT ABUTMENT

LINE "C"  
CENTERLINE OF CUTOFF TRENCH

CUTOFF TRENCH

# ROUTING PLAN - LEFT ABUTMENT

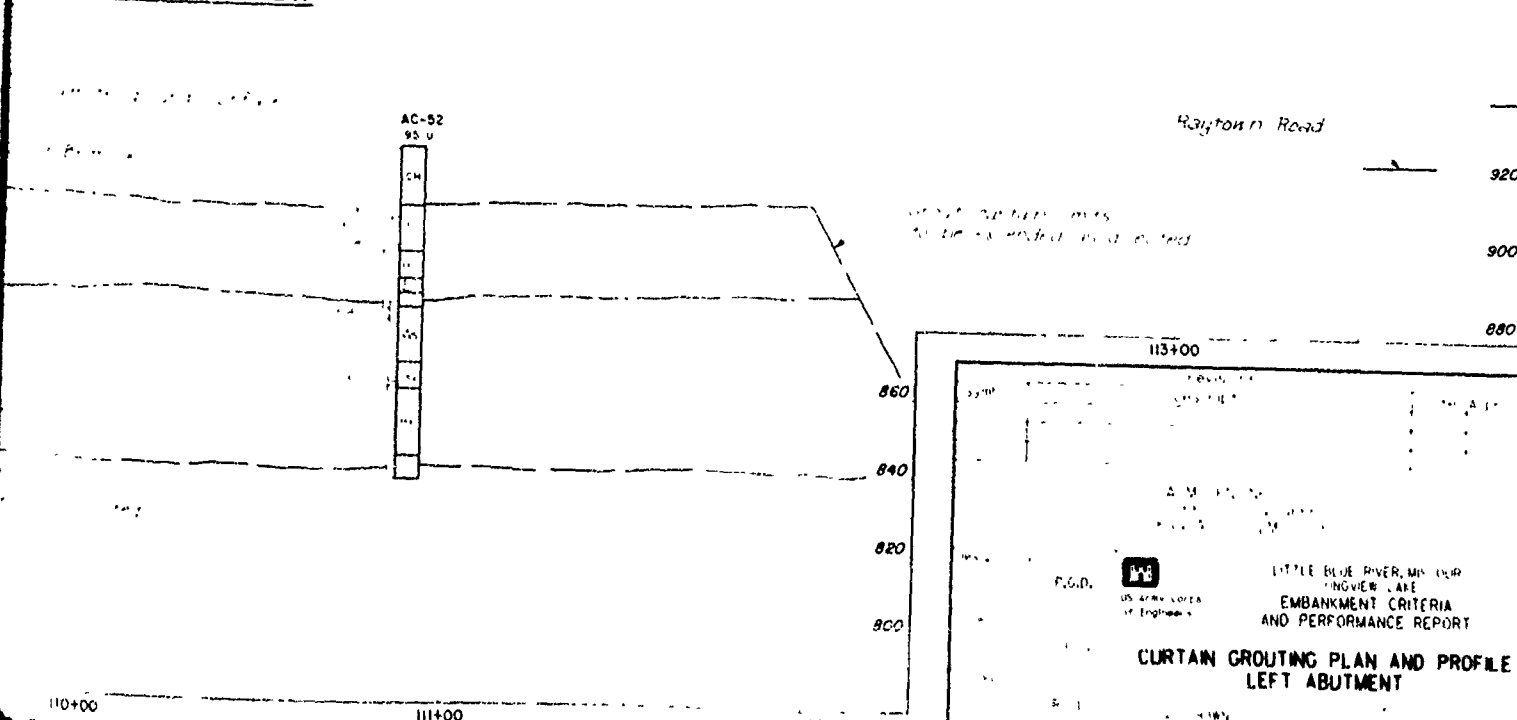


## GROUTING PROFILE - LEFT ABUTMENT

### NOTES

- For General Geologic Column and Legend, See Plate No. 6
- For plan of Borings, See Plate No. 3
- For boring logs showing soil and bedrock description, See Plate No. 7
- Only primary holes are shown on plans. Within the limits of the cutoff trench, secondary grout holes shall be mandatory.
- Indicates direction of grout holes (Direction may be changed by the Contracting Officer)

## ROUTING PLAN - FAR LEFT ABUTMENT



## GROUTING PROFILE - FAR LEFT ABUTMENT

F.S.D.  
U.S. ARMY CORPS  
OF ENGINEERS

LITTLE BLUE RIVER, MINOR  
INGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

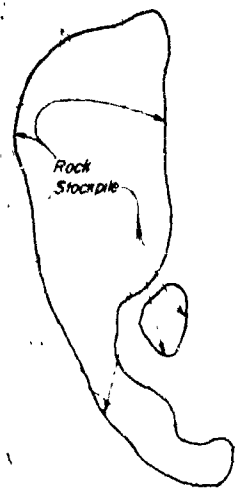
CURTAIN GROUTING PLAN AND PROFILE  
LEFT ABUTMENT

RBL-3-1416



NOTE: See Plate No. 18 for actual contours of left abutment and embankment.

See clarification drawing  
LTR Nov 4, 1982



for actual  
abutment and

See clarification drawing  
LTR Nov 4, 1982

LEFT  
ABUTMENT  
EXCAVATION  
AND BACKFILL

Repaired Sewer  
(Existing)

Ⓔ Roadway

STA. 61+00

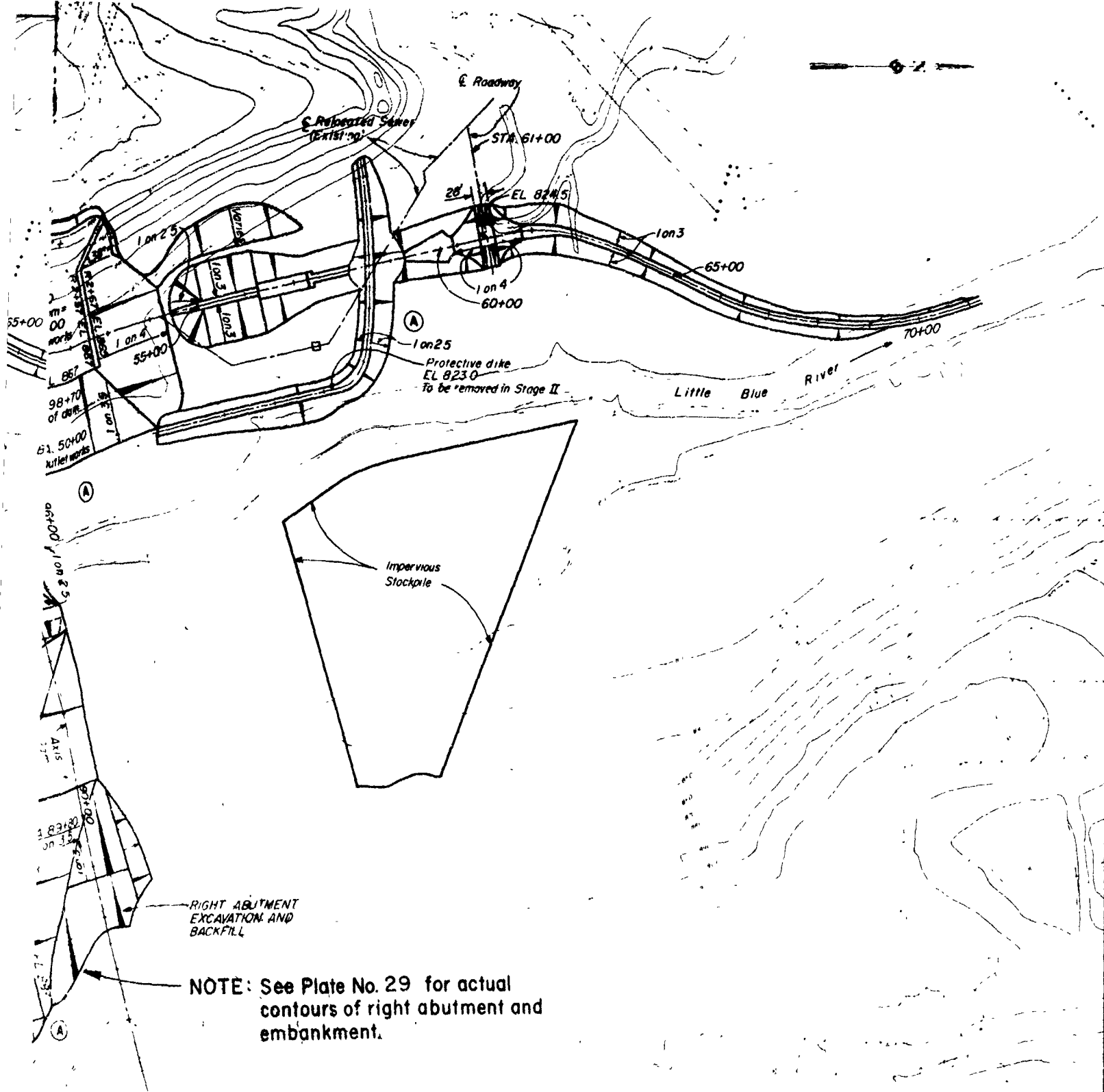
EL. 824.5

60+00

Protective dike  
EL. 823.0  
To be removed in Stage II

Impervious  
Stockpile

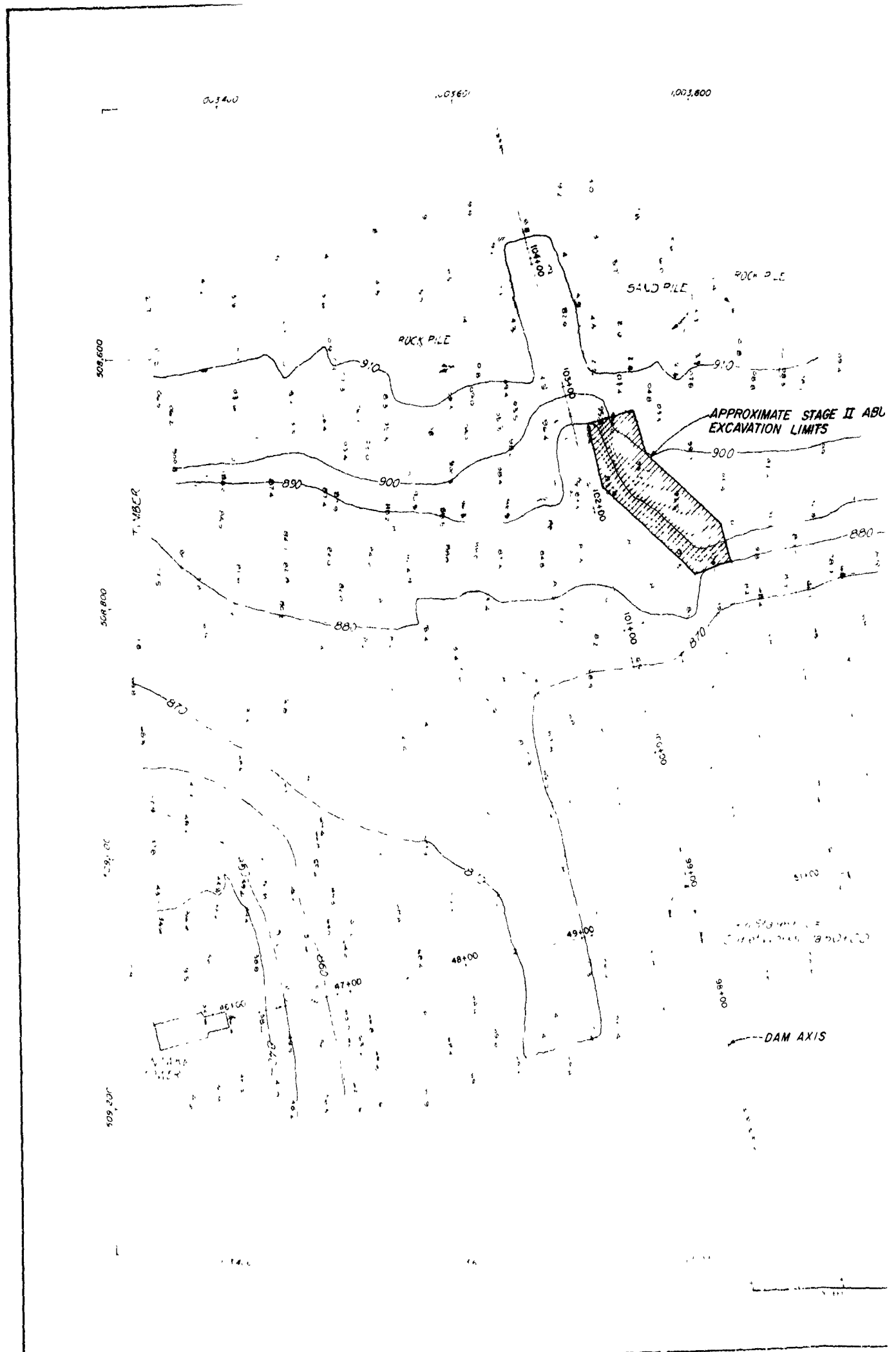
NOTE: See Plate No. 29 for actual  
contours of right abutment and  
embankment.



Revisions			
Symbol	Descriptions	Date	Approved
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	R.G.D.	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by:	J.W.T.	<b>EMBANKMENT AT COMPLETION            OF STAGE I</b>	
Checked by:	R.G.D.		
	R.G.D.		
	Scale AS SHOWN	Sheet number:	Plot Scale:
	Date		Design File:
	Dwg. No.		File No. REL-3-1417

LONGVIEW LAKE

PLATE NO. 17



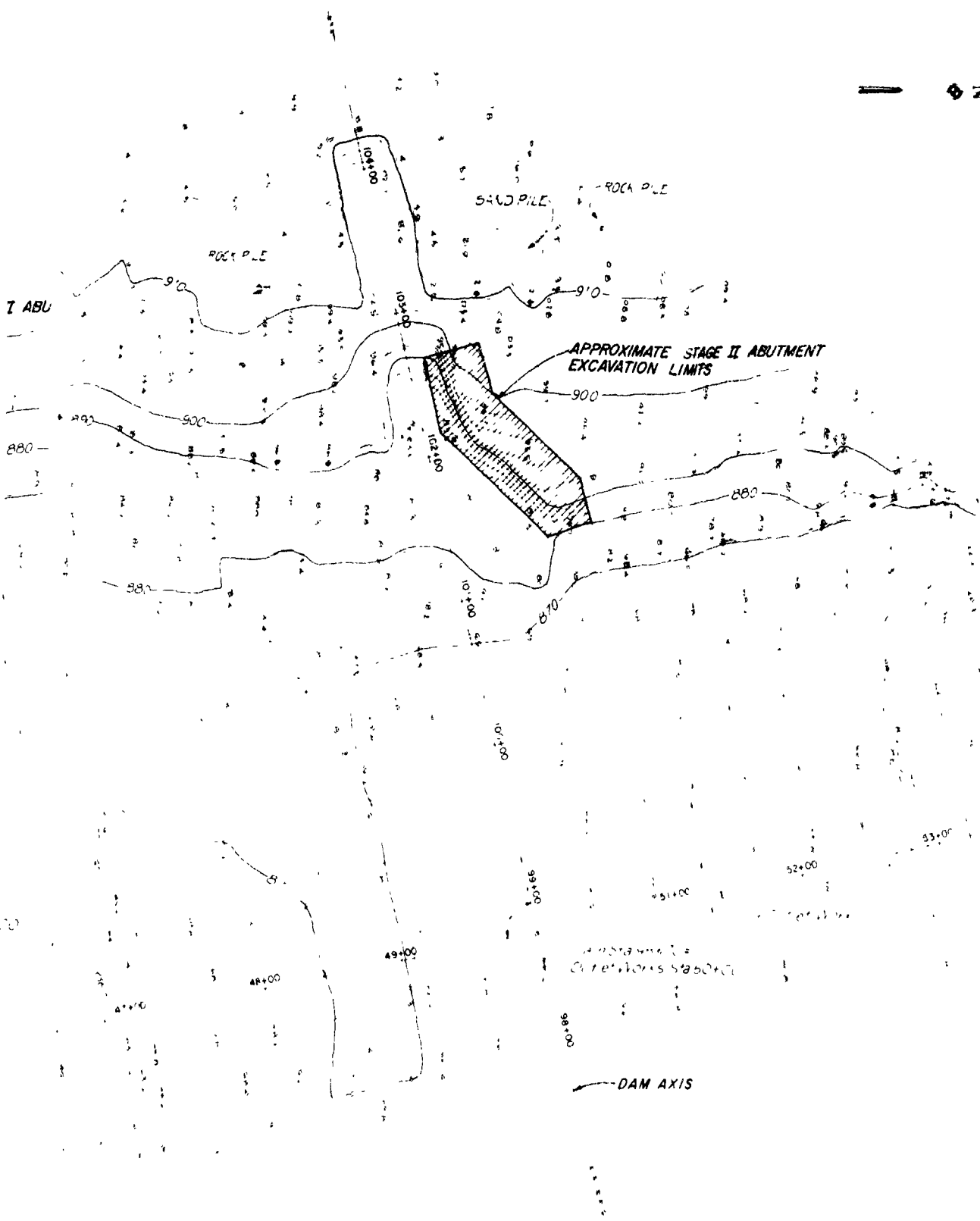
1003,600

1003,800

1004,000

1004,200

14



Notes

- 1. Limits of Stage II excavation shown are approximate. A full vertical and horizontal survey of the area should be made and the limits of excavation should be directed by the Contract Engineer.
- 2. Existing Stage I rock surfaces should be foundation cleanup or preparation as directed by the Contract Engineer.
- 3. Areas within the Stage II excavation limits should be excavated to depths as directed by the Contract Engineer, and any other materials as determined by the Contract Engineer.

CORPS OF ENGINEERS

1,004,000

1,004,200

1,004,400

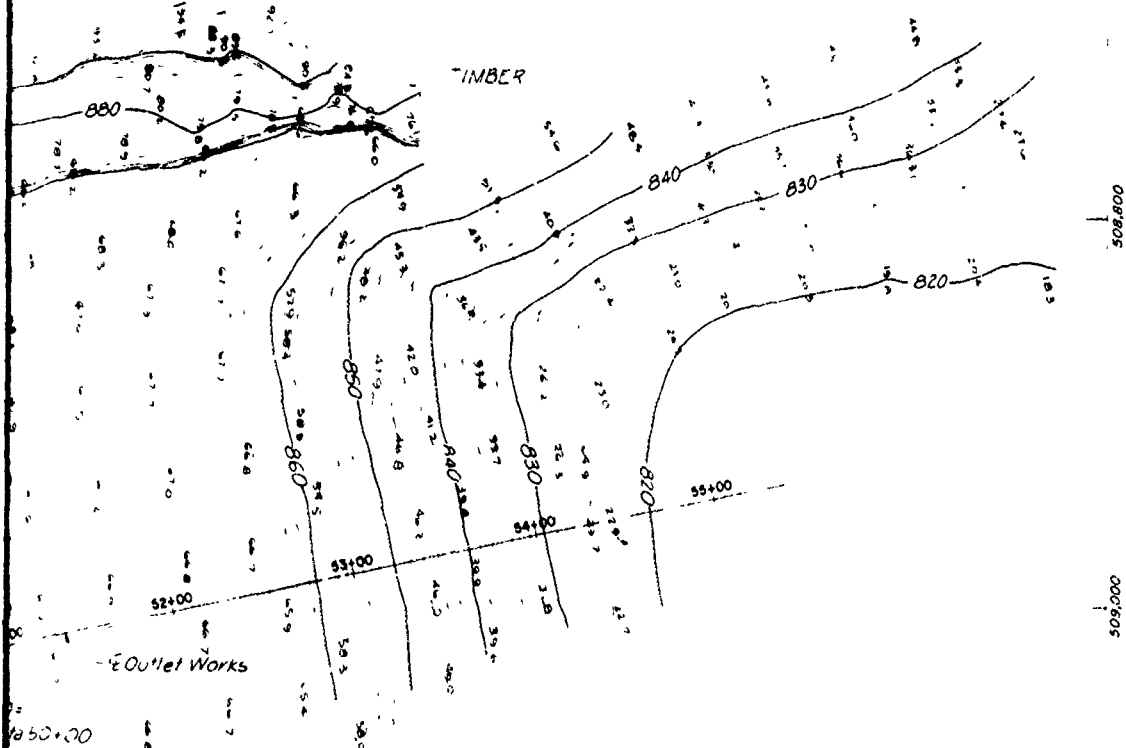


**Notes:**

- 1 Limits of Stage II excavation shown are approximate. Actual vertical and horizontal limits will depend upon conditions encountered and shall be as directed by the Contracting Officer.
- 2 Existing Stage I rock surfaces shall receive foundation cleanup or preparation as directed by the Contracting Officer and removal of temporary protective soil cover.
- 3 All areas within the Stage II excavation limits shall be excavated to depths as directed, which will remove all highly organic material, loose rock and any other objectionable foundation materials as determined by the Contracting Officer.
- 4 Sand and rock piles to be removed by Stage I contractor.

**STAGE II ABUTMENT LIMITS**

TIMBER



CORPS OF ENGINEER'S SURVEY-MARCH, 1982


1,004,000

1,004,200

1,004,400

Revisions			
Symbol	Descriptions	Date	Approved

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: R.G.D.  LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

DRAWN BY: T.L.G.

CHECKED BY: R.G.D.

SUBMITTED BY: R.G.D.

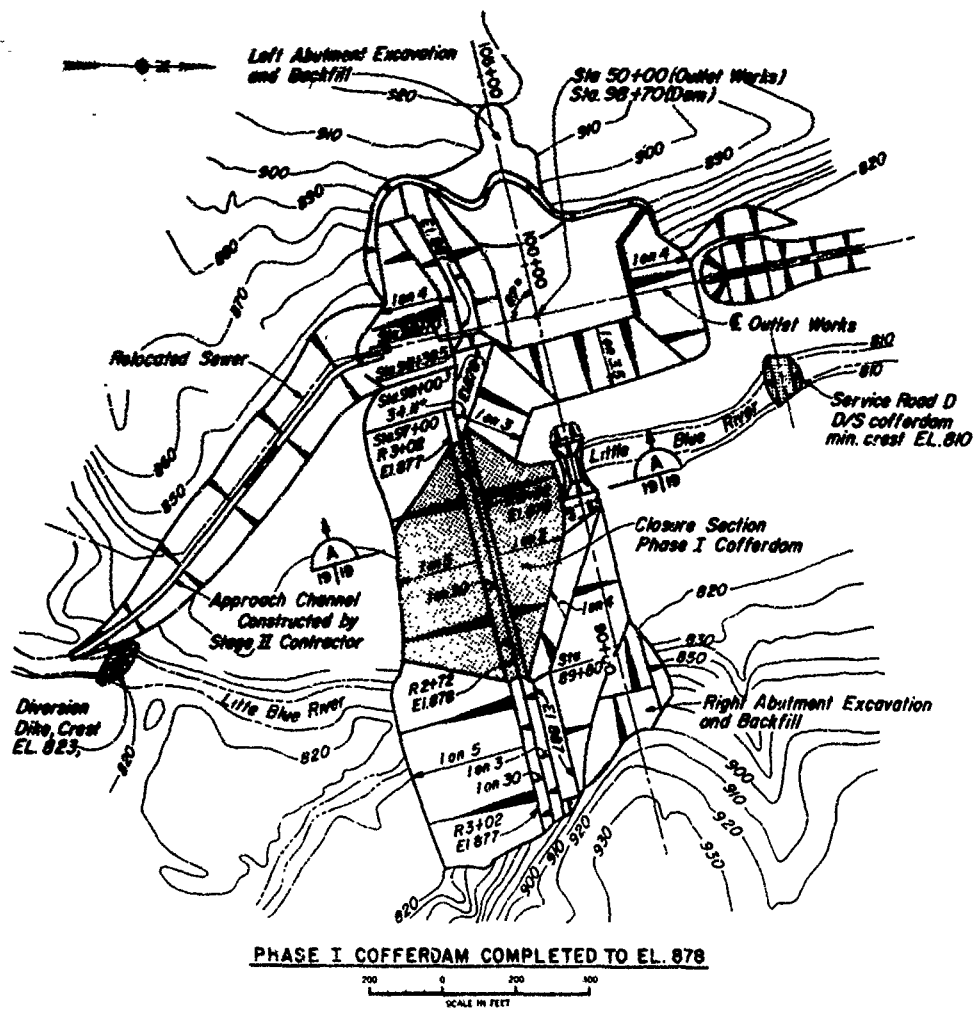
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Sheet Number:  

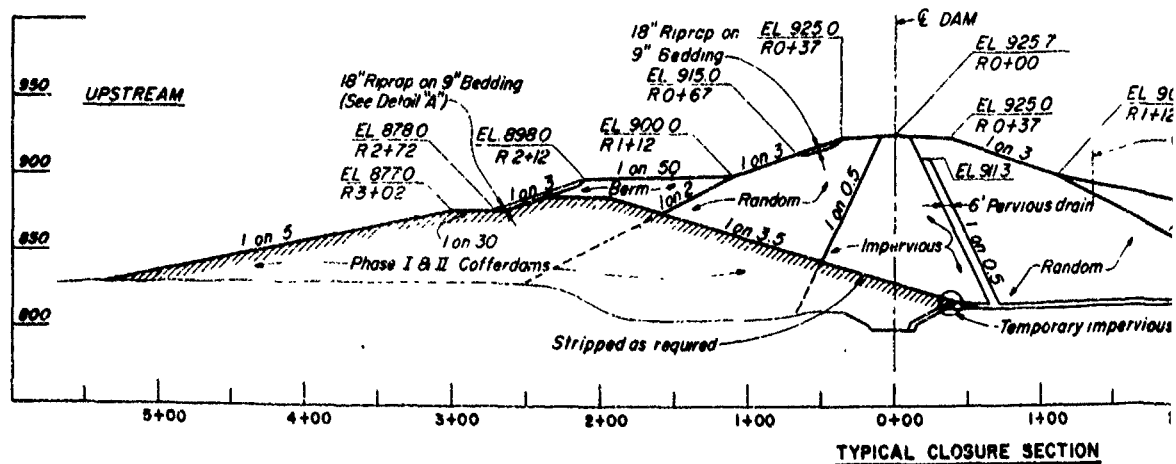
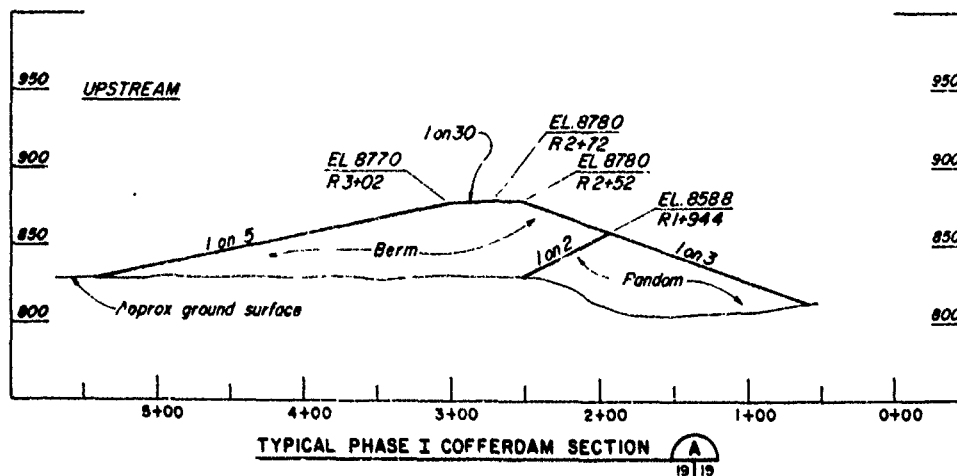
Plot Scale:  

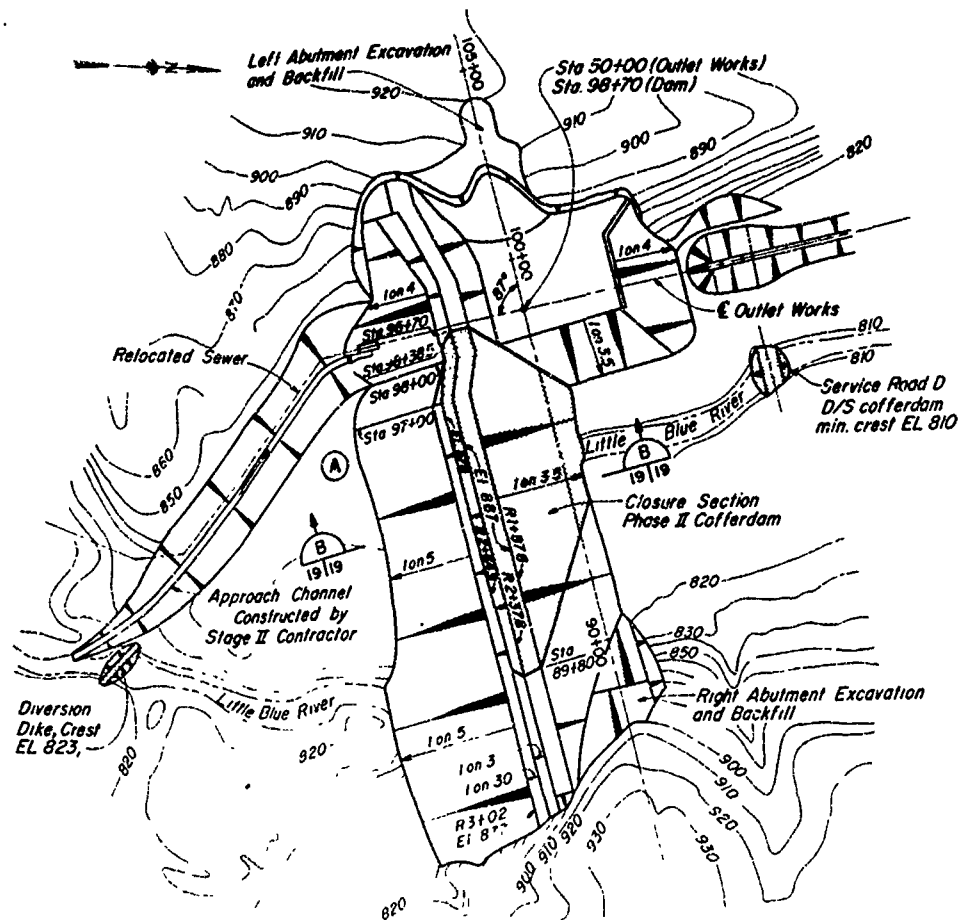
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File No: RBL-3-M18

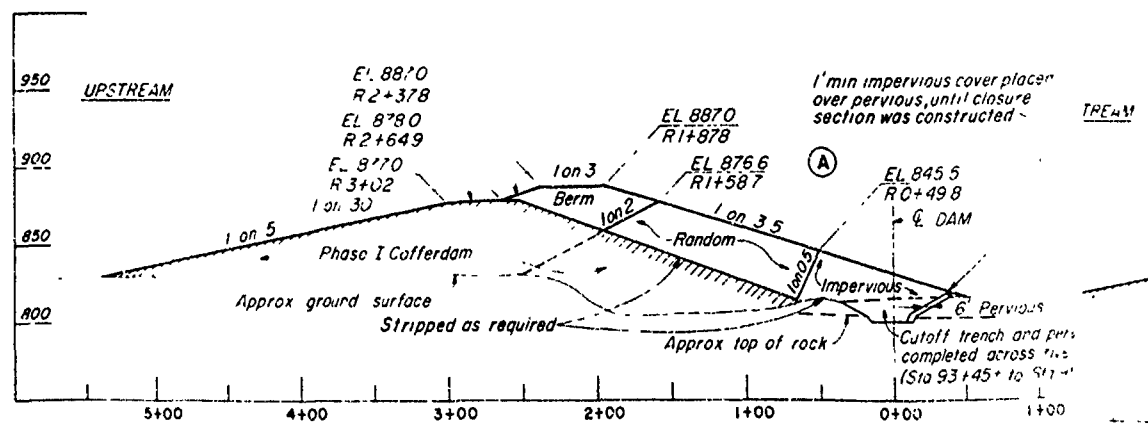


ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929

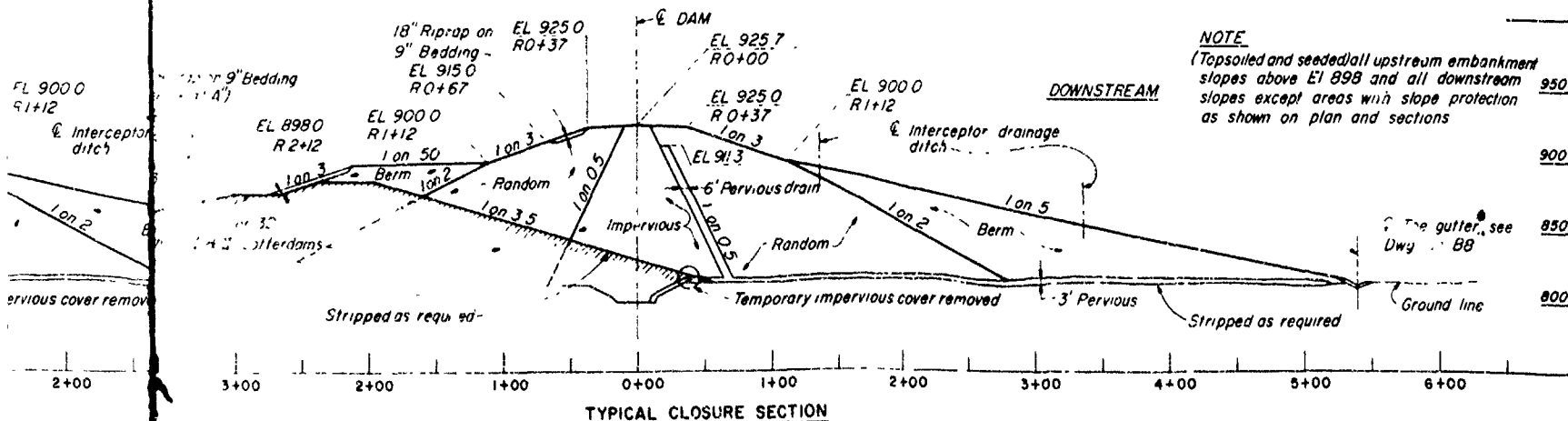




PHASE II COFFERDAM COMPLETED TO EL. 887

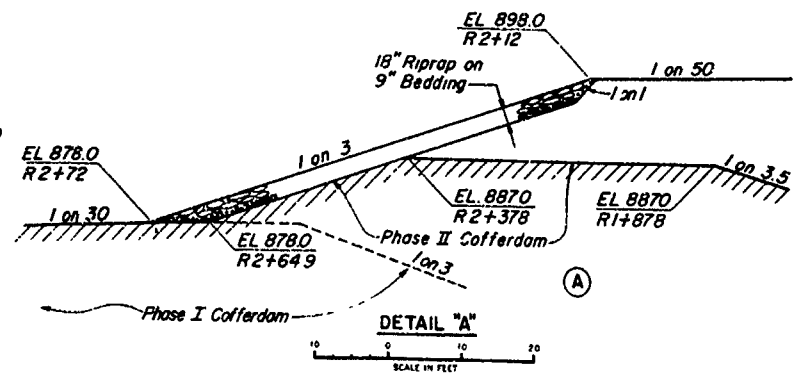
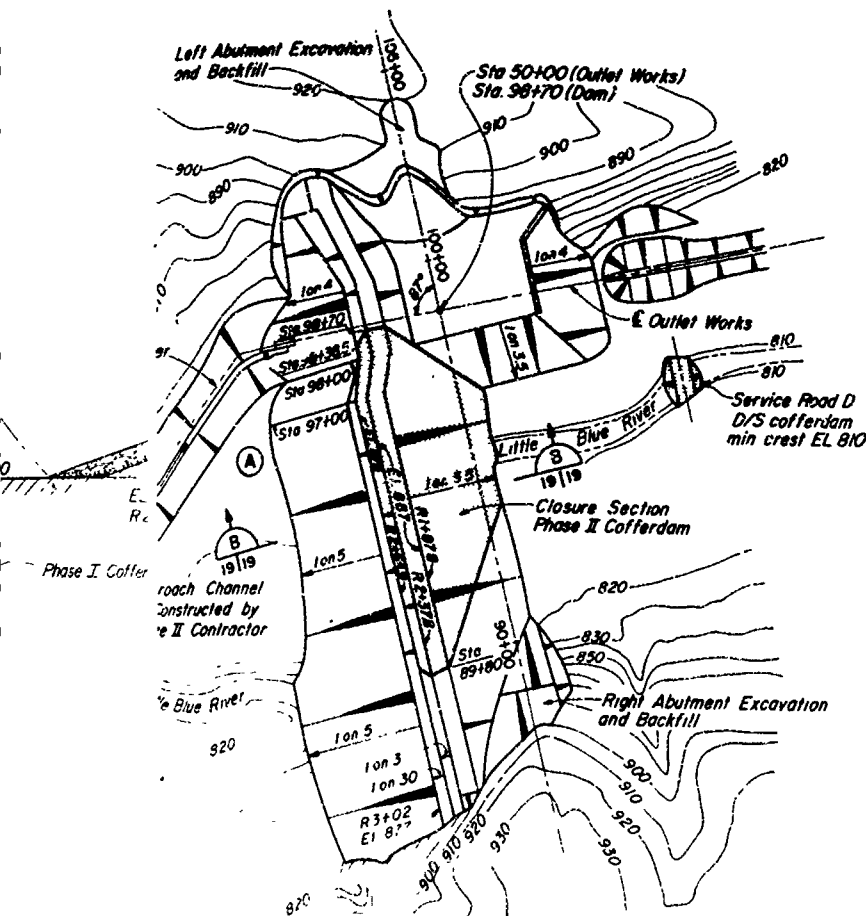


TYPICAL PHASE II COFFERDAM SECTION B  
19 | 19



NOTE  
(Topsoiled and seeded) all upstream embankment slopes above El 898 and all downstream slopes except areas with slope protection as shown on plan and sections





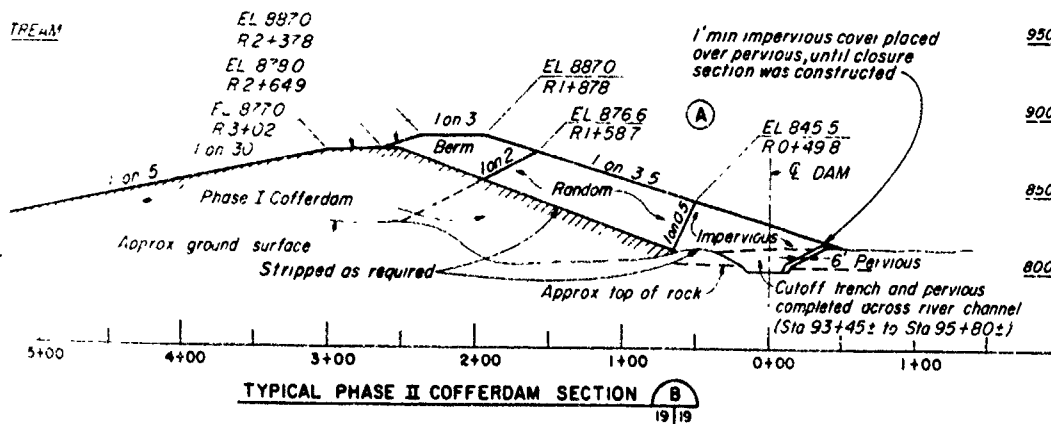
PHASE II COFFERDAM COMPLETED TO EL. 887

ous cover placed  
until closure  
constructed -

TREAM

45.5  
49.8  
DAM

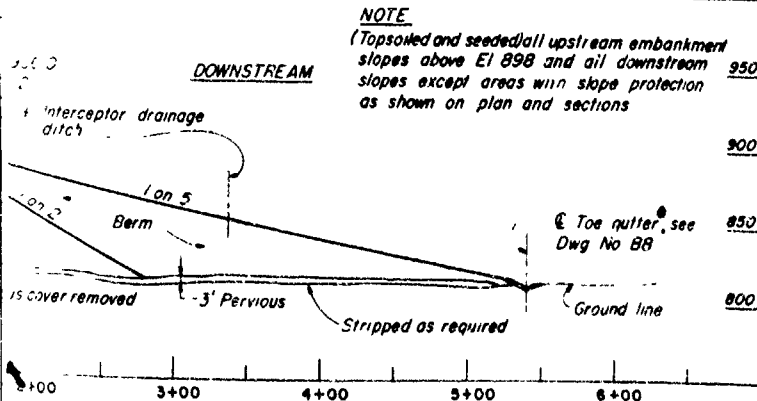
6' Pervious  
cutoff trench and pervious  
completed across river  
Sta 93+45± to Sta 95+80±



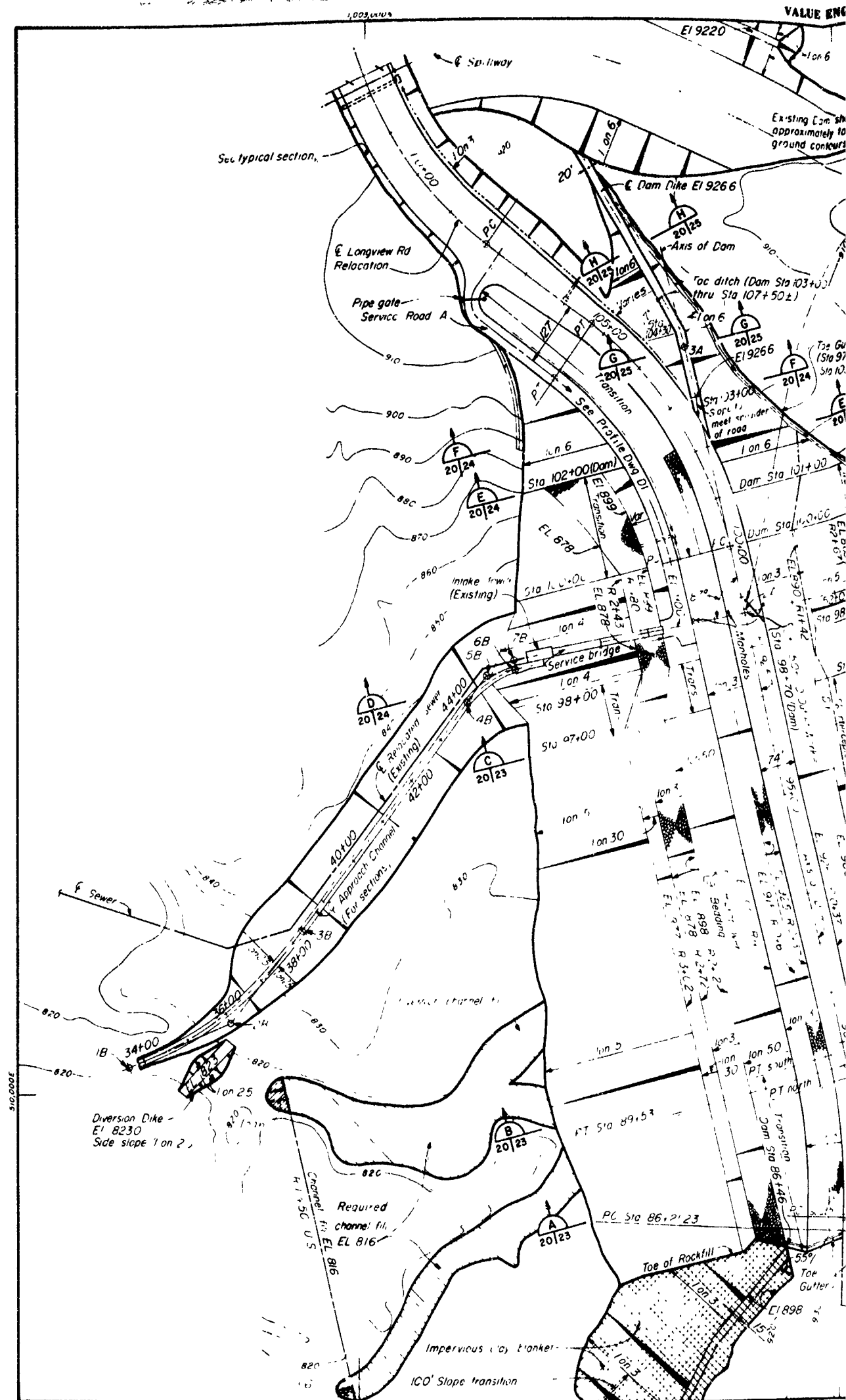
TYPICAL PHASE II COFFERDAM SECTION B

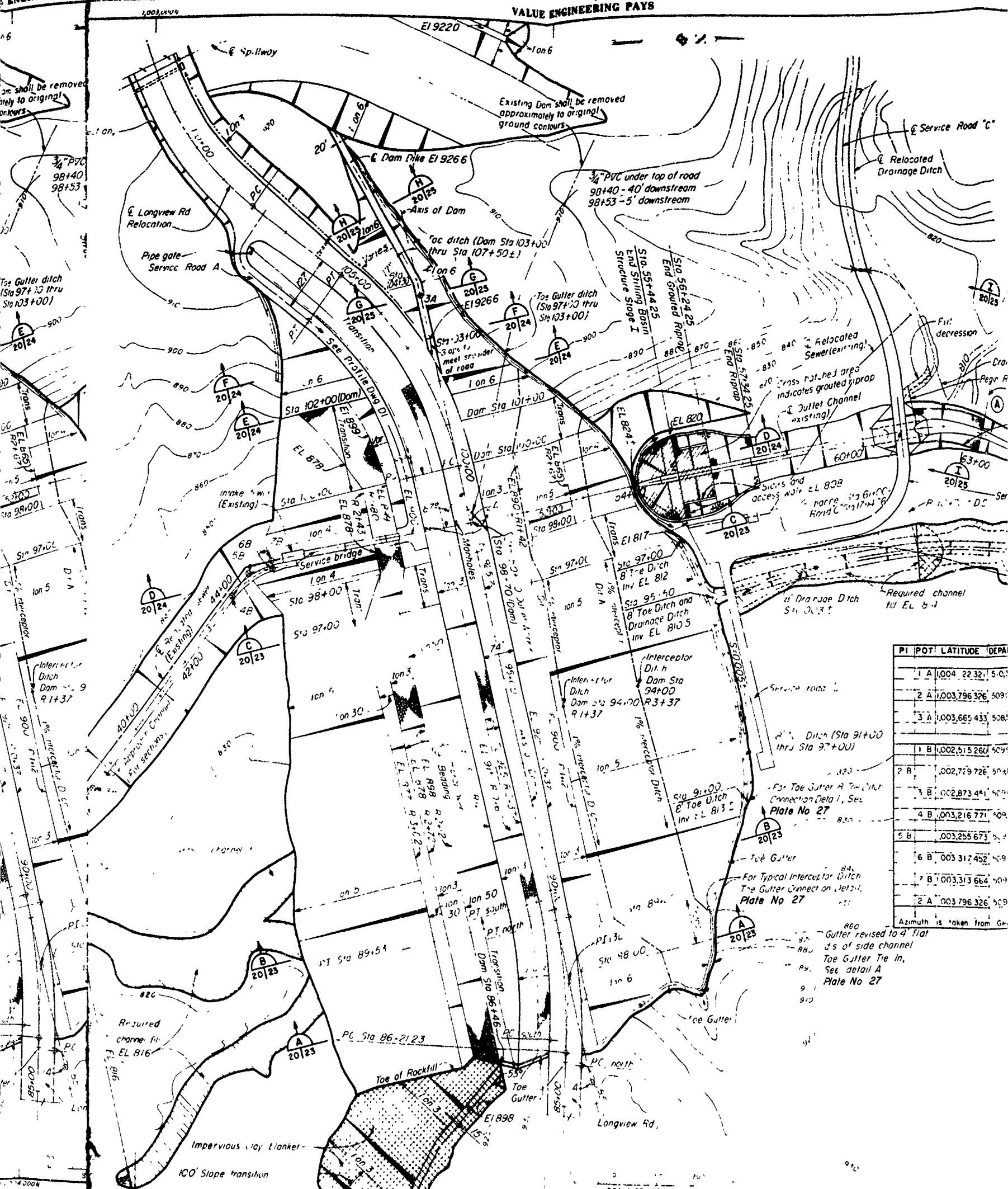
NOTE

(Topsoiled and seeded) all upstream embankment  
slopes above EL 898 and all downstream  
slopes except areas with slope protection  
as shown on plan and sections



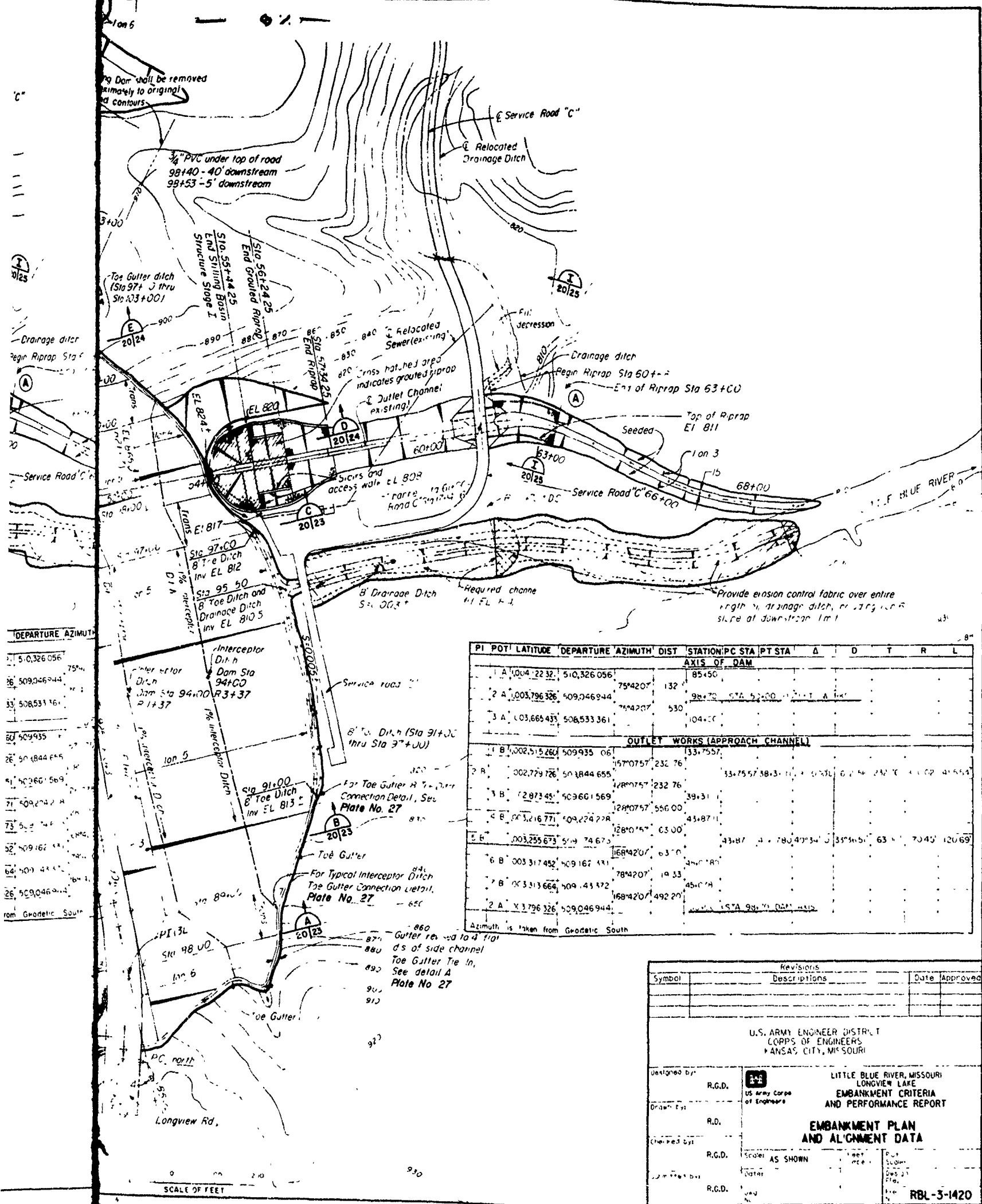
Revisions			
Symbol	Descriptions	Date	Approved
U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by R G D.		LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT DIVERSION AND CLOSURE: PLANS AND SECTIONS	
Drawn by S L M		Scale AS SHOWN	Sheet number
Checked by R G D / W B E		Date	
Submitted by R C U		Dwg No	File No

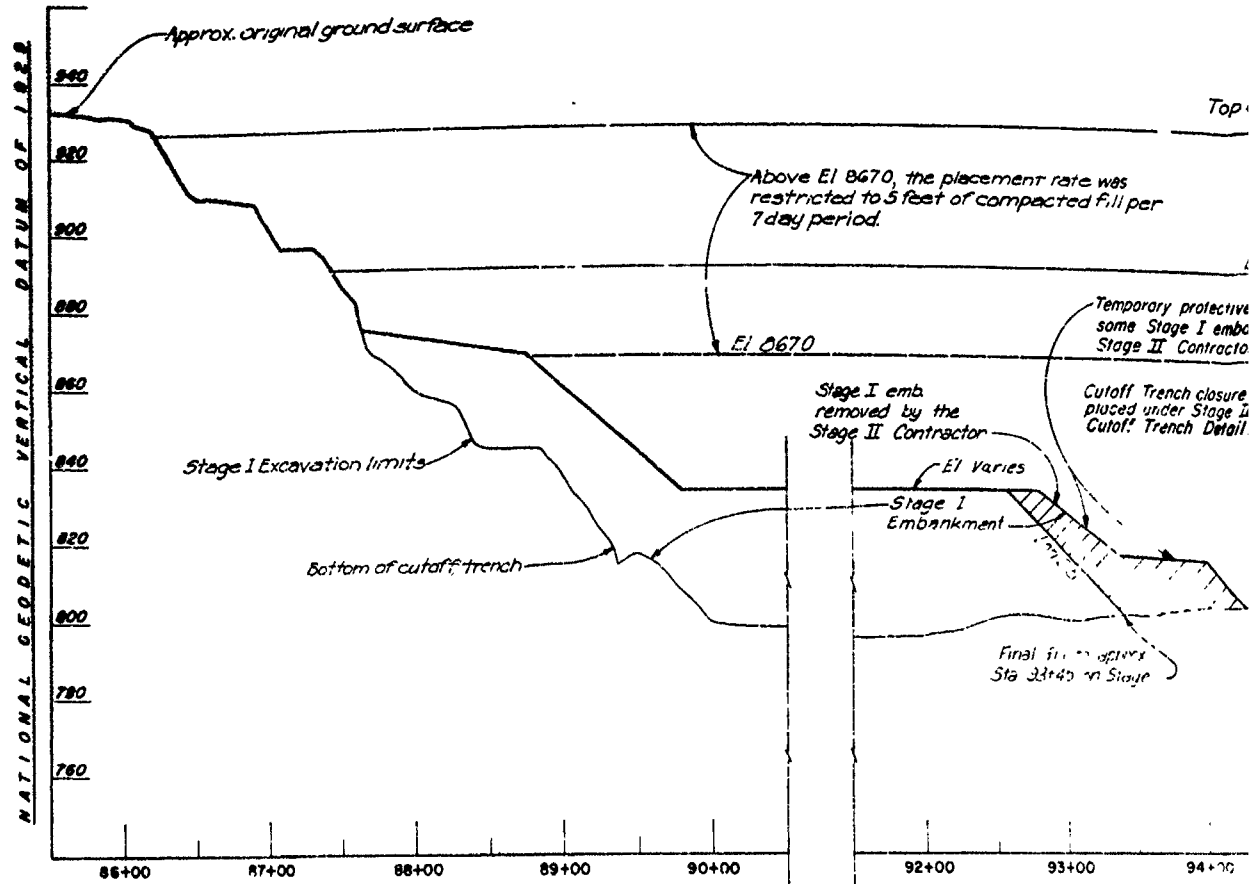




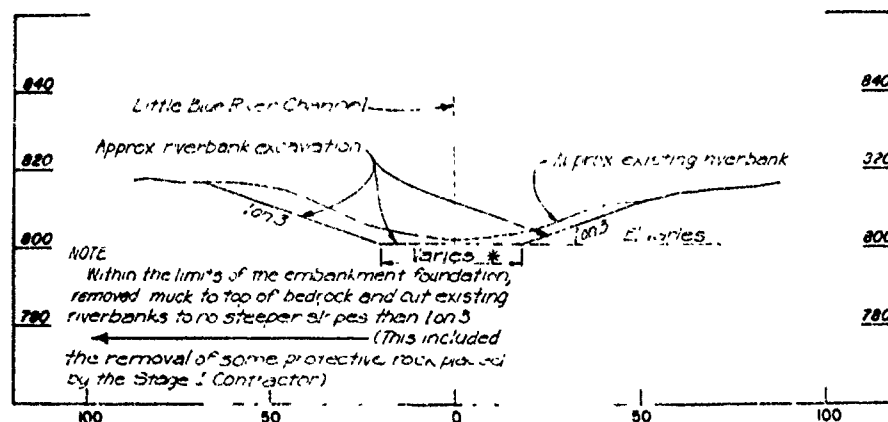
PI	POT	LATITUDE	DEPART
1	A	1,004,223.21	5,013.00
2	A	1,003,796.326	5,091.00
3	A	1,003,665.433	5,081.00
1	B	1,002,515.260	5,091.00
2	B	1,002,729.726	5,091.00
3	B	1,002,873.441	5,091.00
4	B	1,003,216.771	5,091.00
5	B	1,003,255.673	5,091.00
6	B	1,003,317.452	5,091.00
7	B	1,003,313.664	5,091.00
2	A	1,003,796.326	5,091.00

Azimuth is taken from Gr.

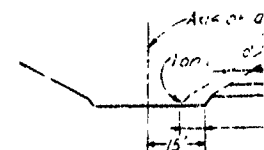




PROFILE ALONG DAM  
LOOKING



TYPICAL SECTION  
RIVERBANK EXCAVATION INCLUDING ABANDONED  
RIVER CHANNEL AND EXISTING CHANNEL



Top of Embankment

Top of Embankment El. 925.7

El. 890 Cont.

El. 890 Construction halt (6 months)

protective plug and so  
I embankment was  
tractor

Closure section  
Stage II contract, see  
Detail

El. 867.0

Stage I emb.  
removed by the  
Stage II Contractor

Temporary protective plug and backfill including  
some Stage I embankment was removed by  
Stage II Contractor

Cutoff Trench closure section  
placed under Stage II contract, see  
Cutoff Trench Detail

El. Varies

Stage I  
Embankment

Final fill to approx  
Sta 93+45 on Stage I

Final fill to approx  
Sta 95+80 on Stage I

Slope to drain

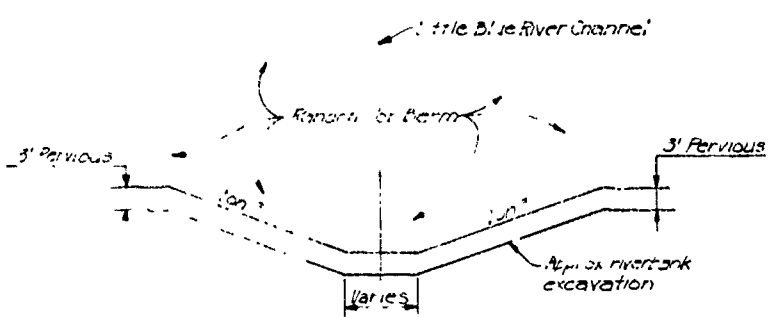
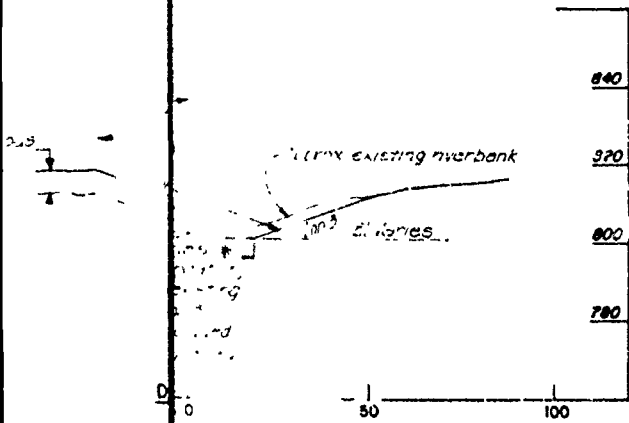
Stage I  
Embankment

Outlet works  
Sta 98+70 at Dam

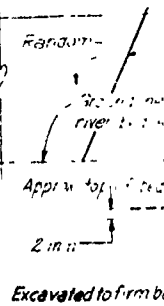
the Blue River Channel

DAM AXIS AND  
LOOKING UPSTREAM

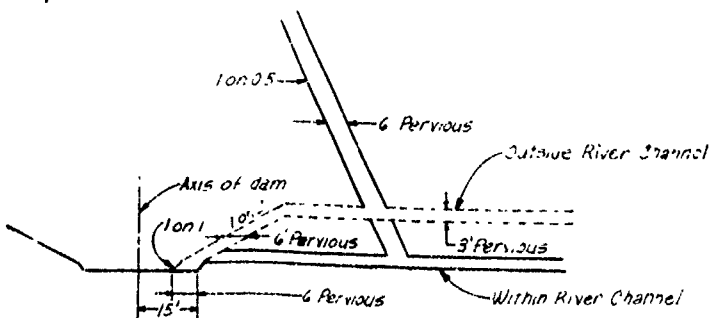
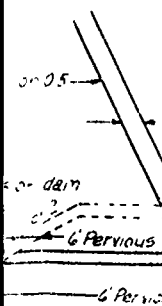
**PROFILE ALONG DAM AXIS AND CUTOFF TRENCH**  
LOOKING UPSTREAM



**TYPICAL SECTION  
RIVER CHANNEL BACKFILL  
DOWNSTREAM OF CUTOFF TRENCH**  
Not to Scale



**SECTION  
INCLUDING ABANDONED  
AND EXISTING CHANNEL**



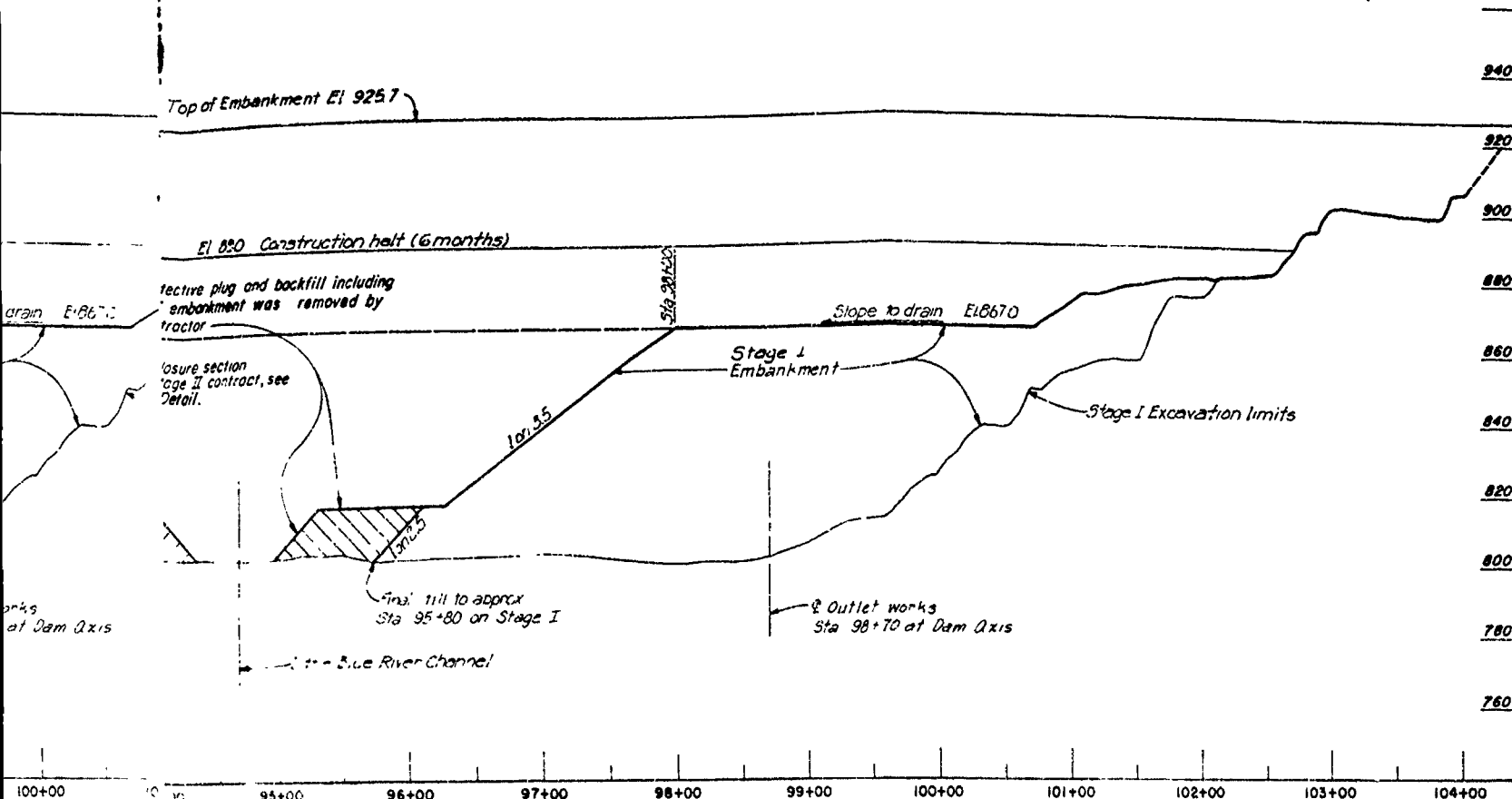
**PERVIOUS TIE-IN CUTOFF TRENCH  
AND RIVER CHANNEL DETAIL**

SCALE IN FEET

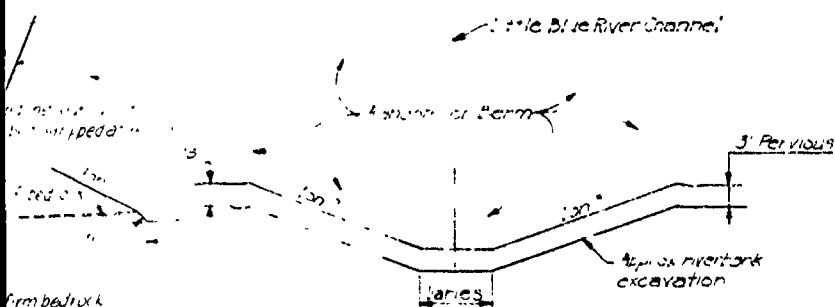
PREVIOUS TIE-IN CUT  
AND RIVER CHANNEL

SCALE IN FEET

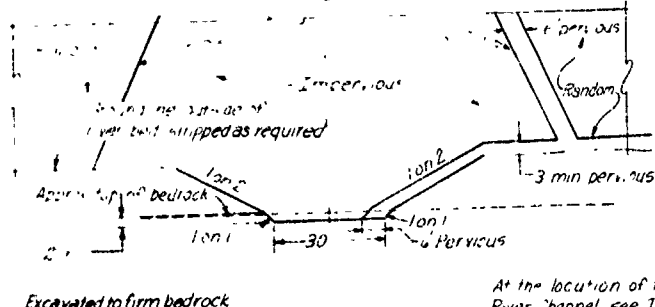
# VALUE ENGINEERING PAYS



## DAM AXIS AND CUTOFF TRENCH LOOKING UPSTREAM

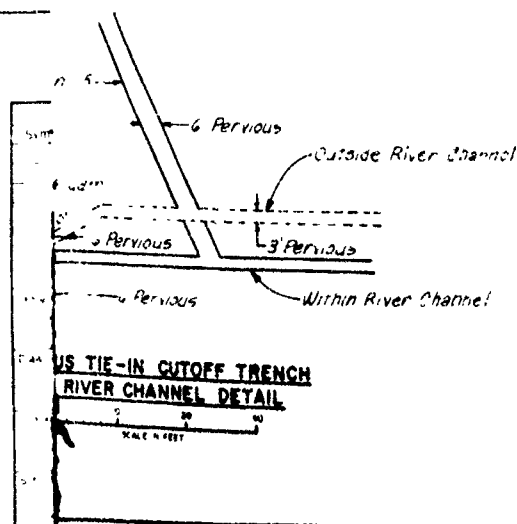


**TYPICAL SECTION  
RIVER CHANNEL BACKFILL  
DOWNSTREAM OF CUTOFF TRENCH**  
Not to Scale

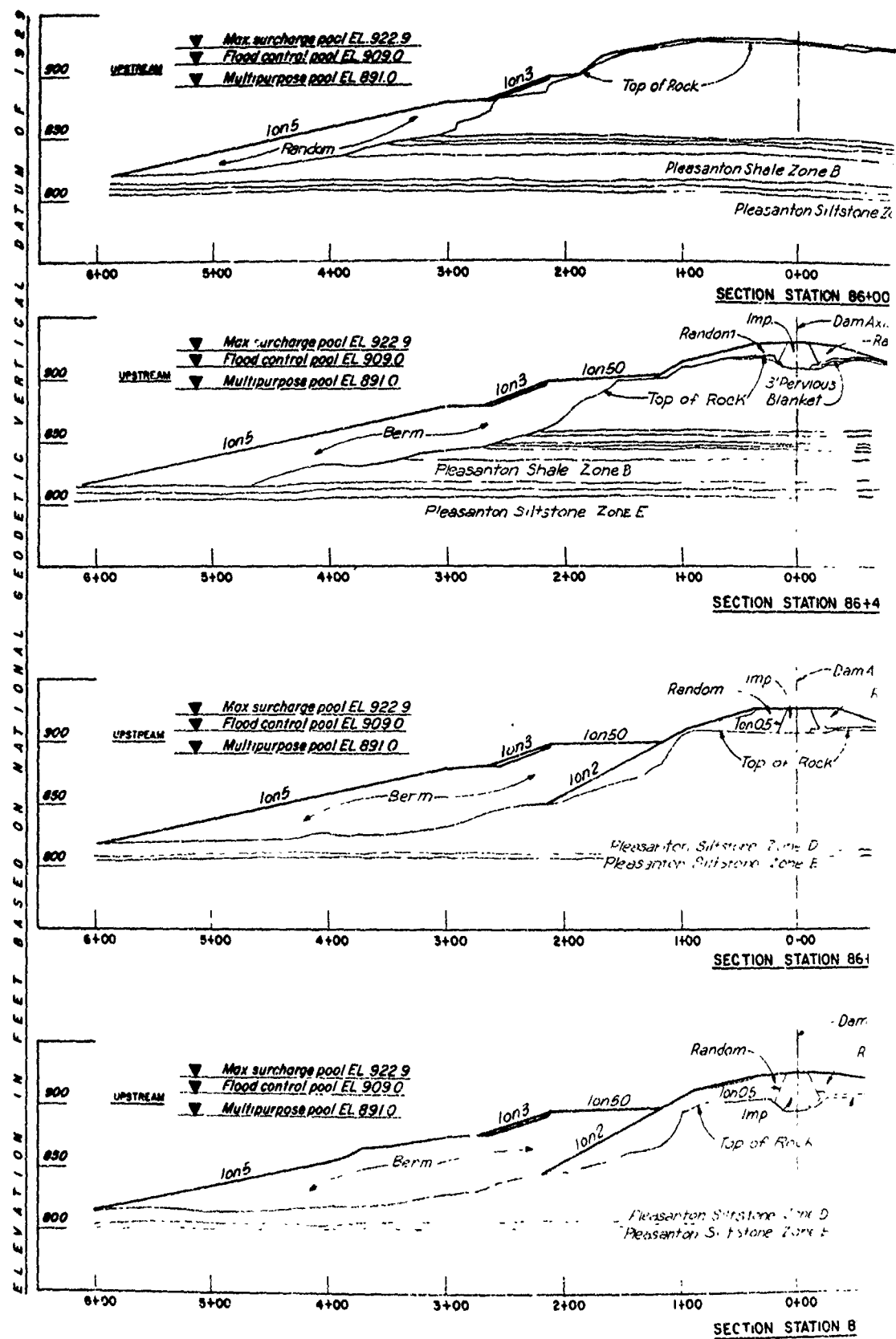


**CUTOFF TRENCH DETAIL  
STA. 93+45± TO STA. 94+80±**

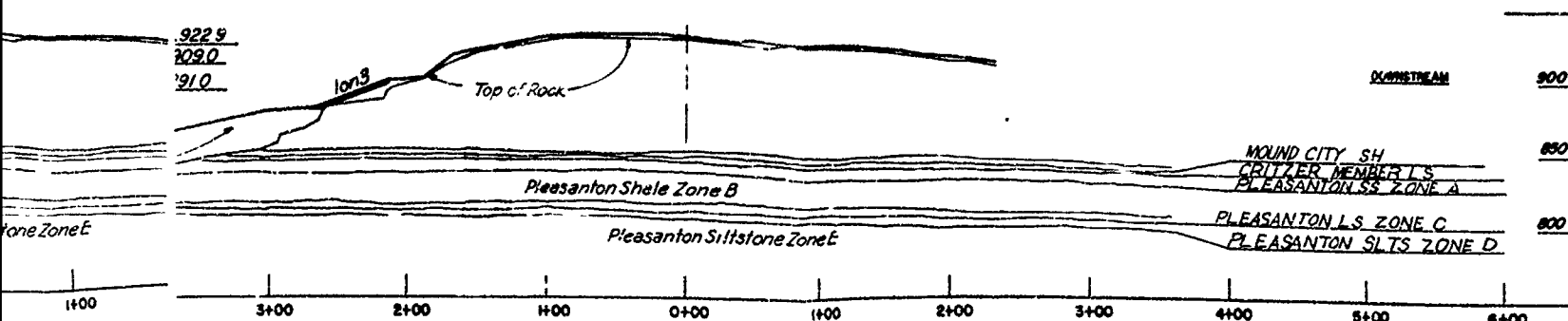
**CUTOFF TRENCH  
STA. 93+45± TO**



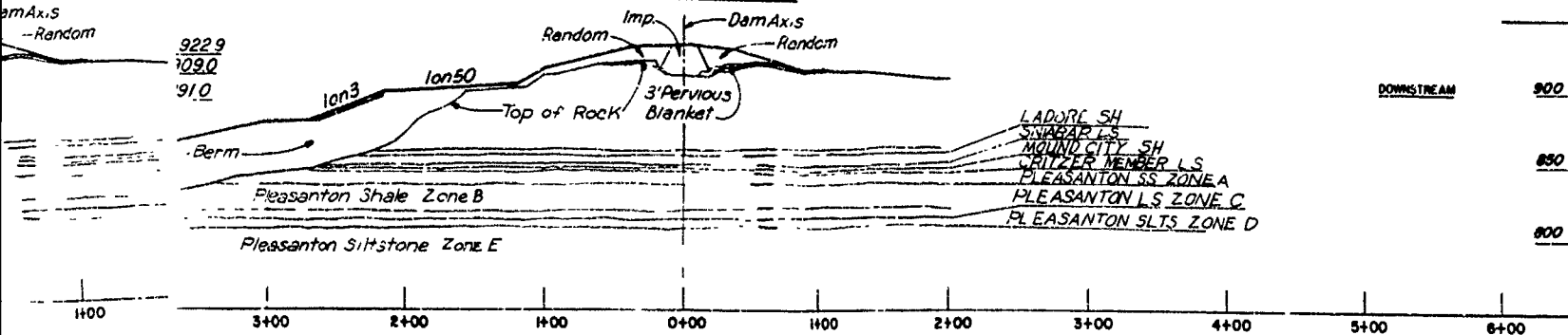
Revisions		Date	Approved
Symbol	Descriptions		
<p align="center"><b>U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</b></p>			
Designed by R G D	<p align="center"><b>U S ARMY CORPS OF ENGINEERS</b></p>	<p align="center"><b>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</b></p>	
Drawn by S L M		<p align="center"><b>DAM AXIS PROFILE AND MISC. SECTIONS &amp; DETAILS</b></p>	
Checked by R G D / W B E		Scale AS SHOWN	Sheet number
Submitted by R G D		Date	File No.
		Dep No.	<b>RBL-3-1421</b>



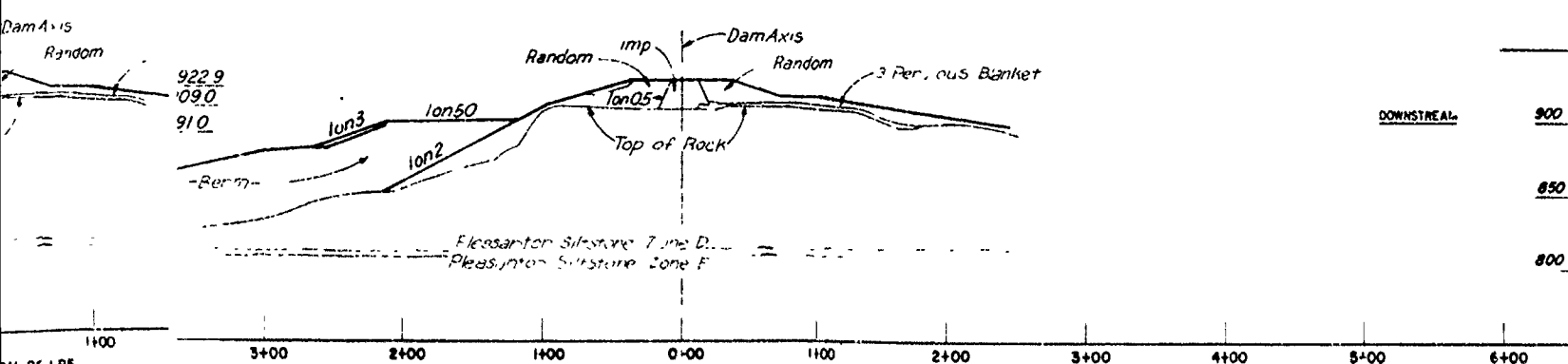




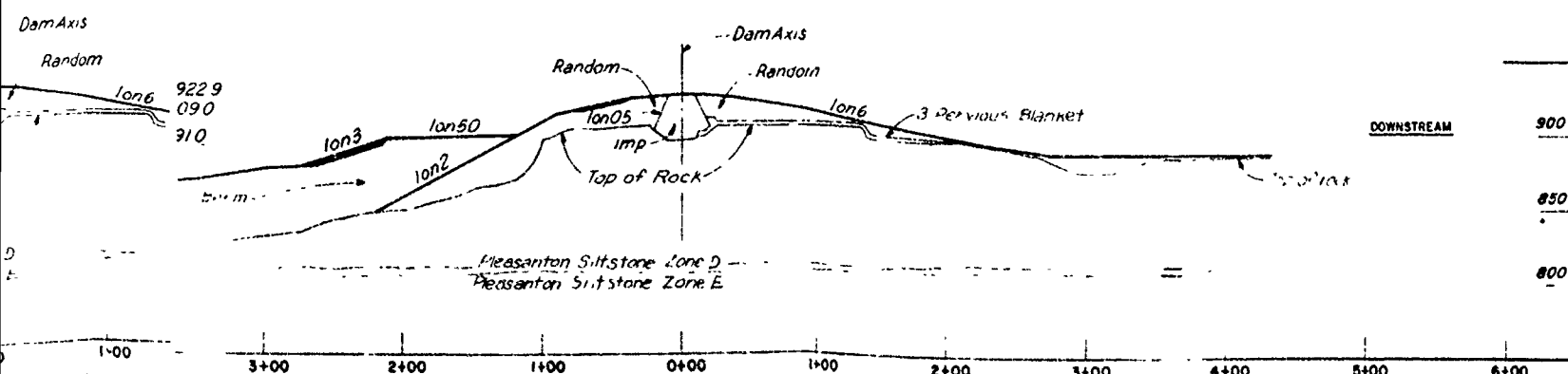
SECTION STATION 86+00



SECTION STATION 86+46

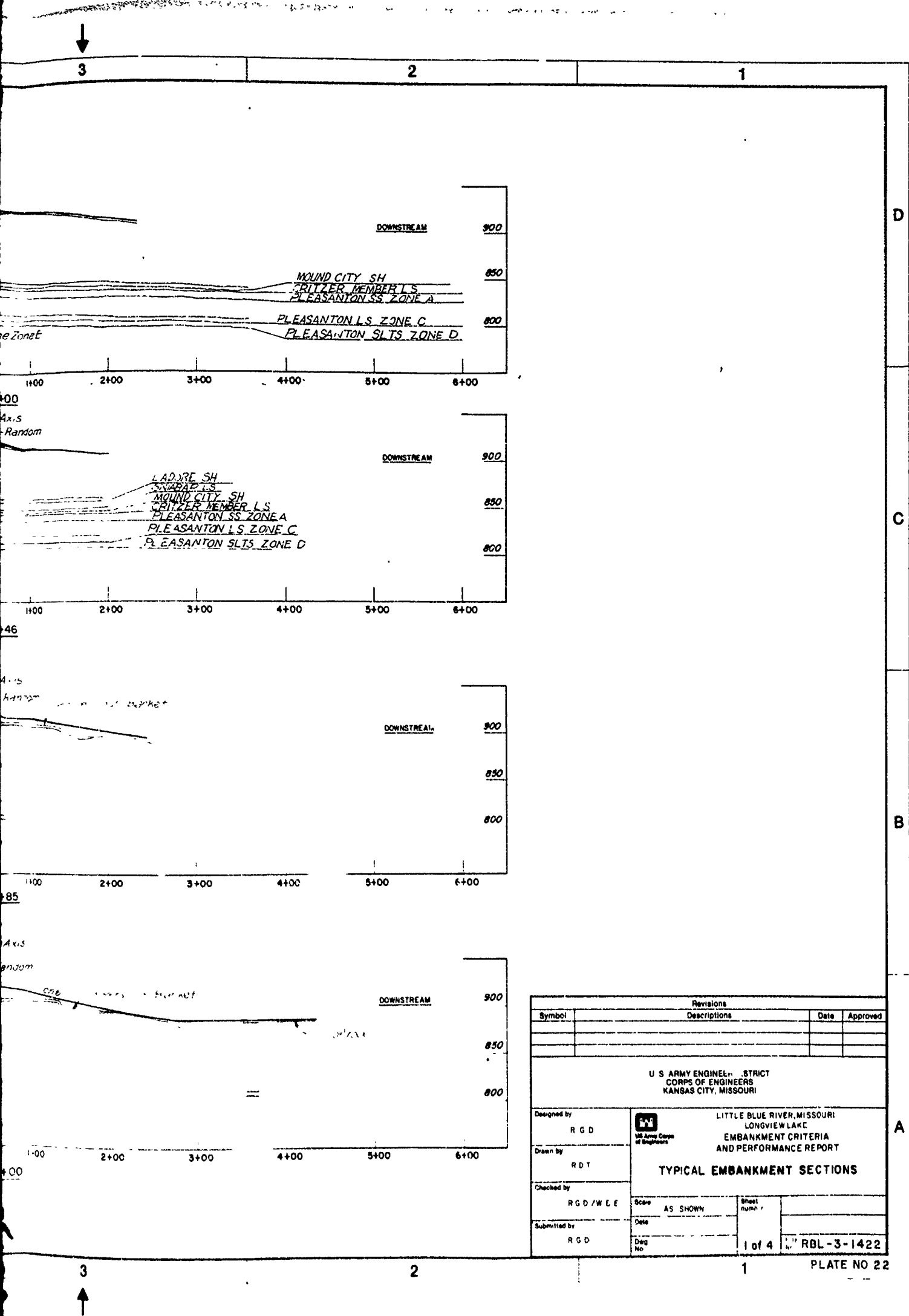


SECTION STATION 86+85



SECTION STATION 87+00

Symbol	
Designed by	R G D
Drawn by	R D T
Checked by	R G D /
Submitted by	R G D



Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by R G D

Drawn by R D T

Checked by R G D / W E E

Submitted by R G D

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

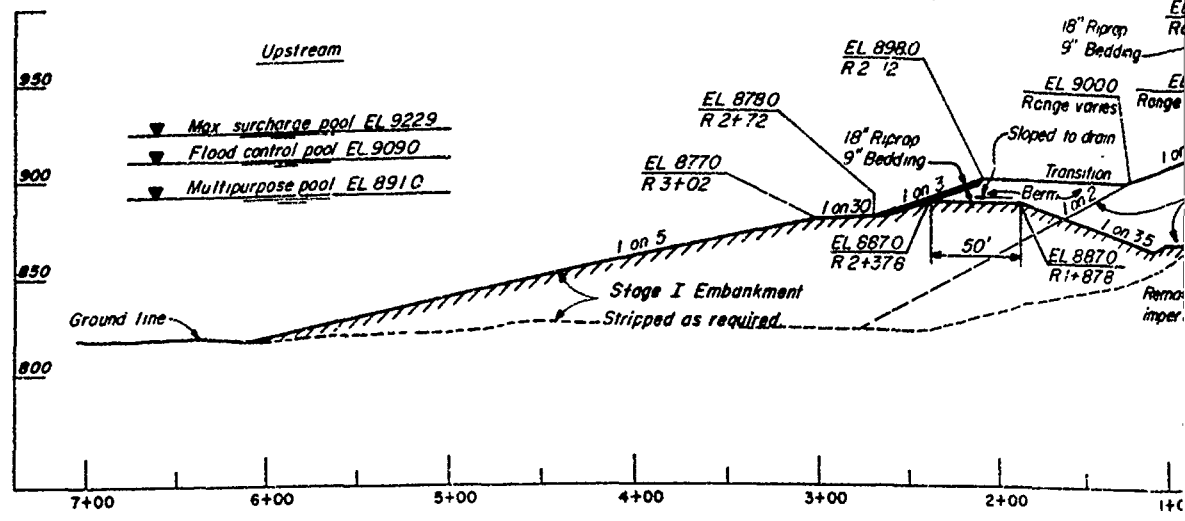
TYPICAL EMBANKMENT SECTIONS

Scale AS SHOWN

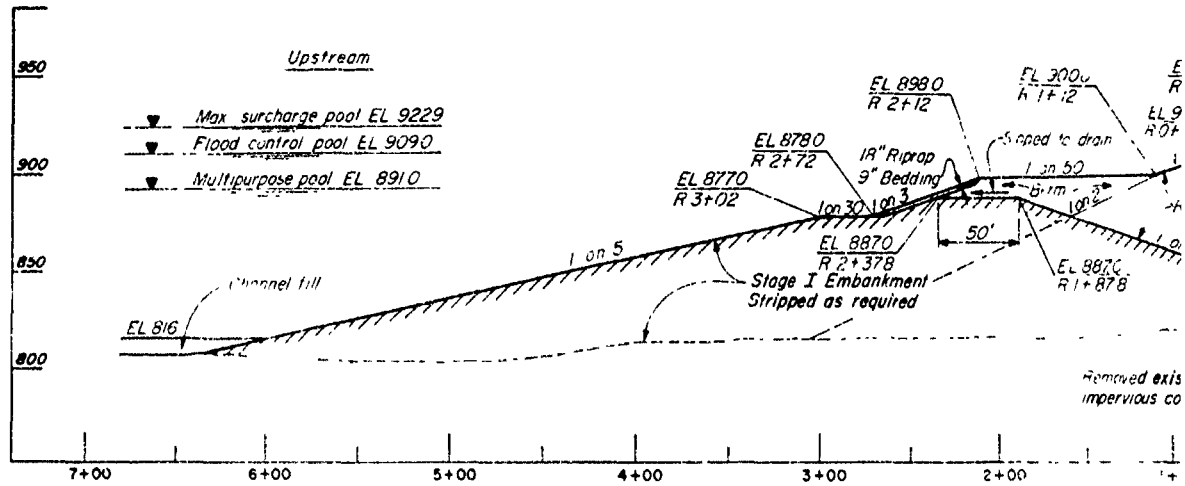
Date

Sheet number 1 of 4

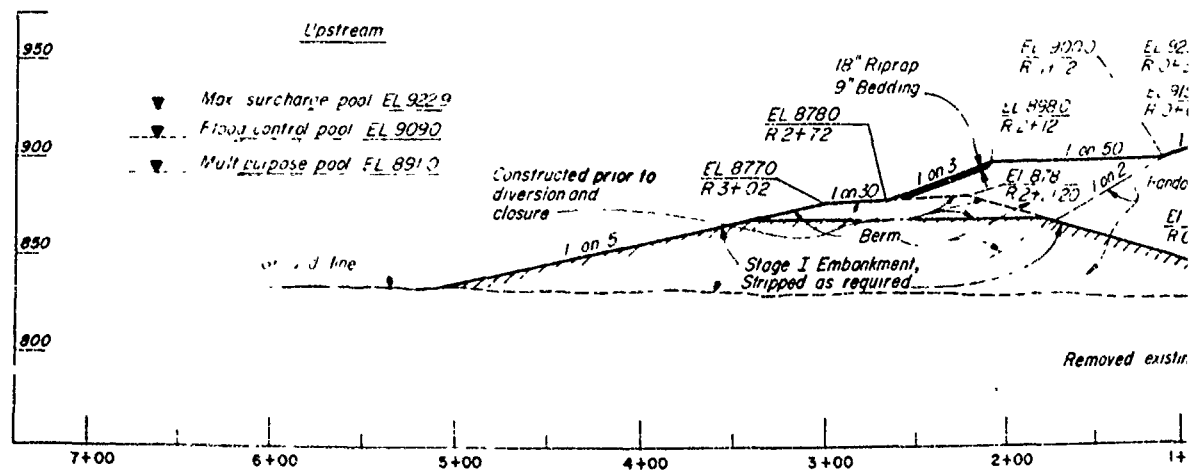
Draw No. RBL-3-1422



RIG



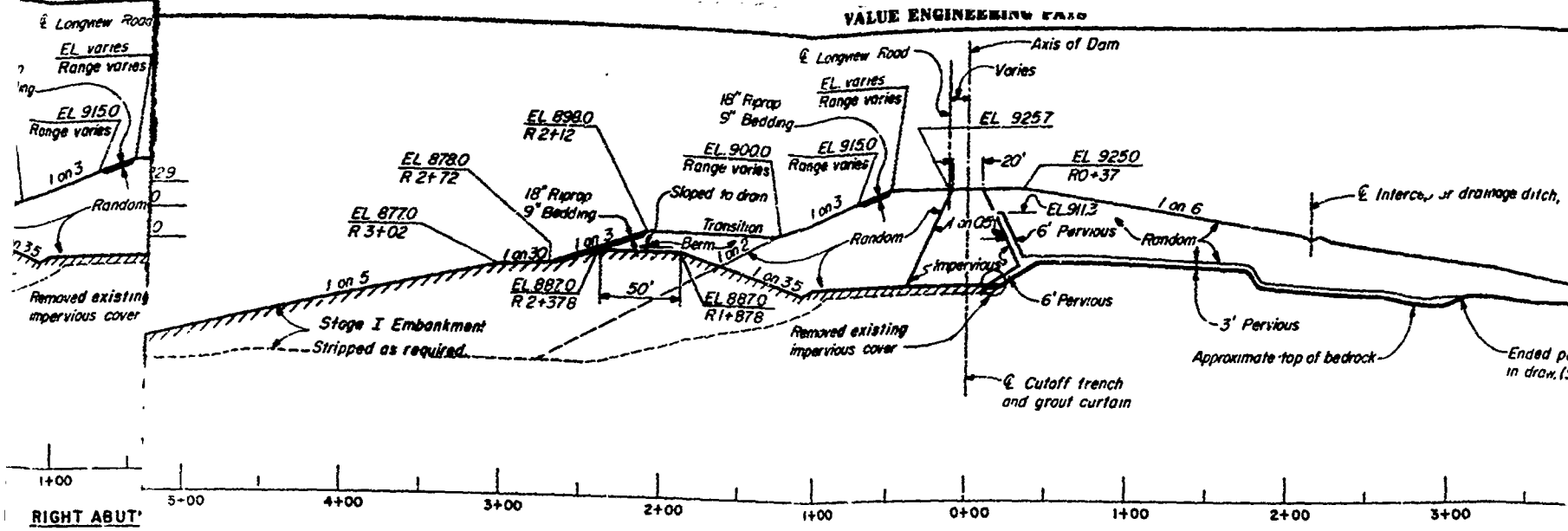
Removed existing impervious core



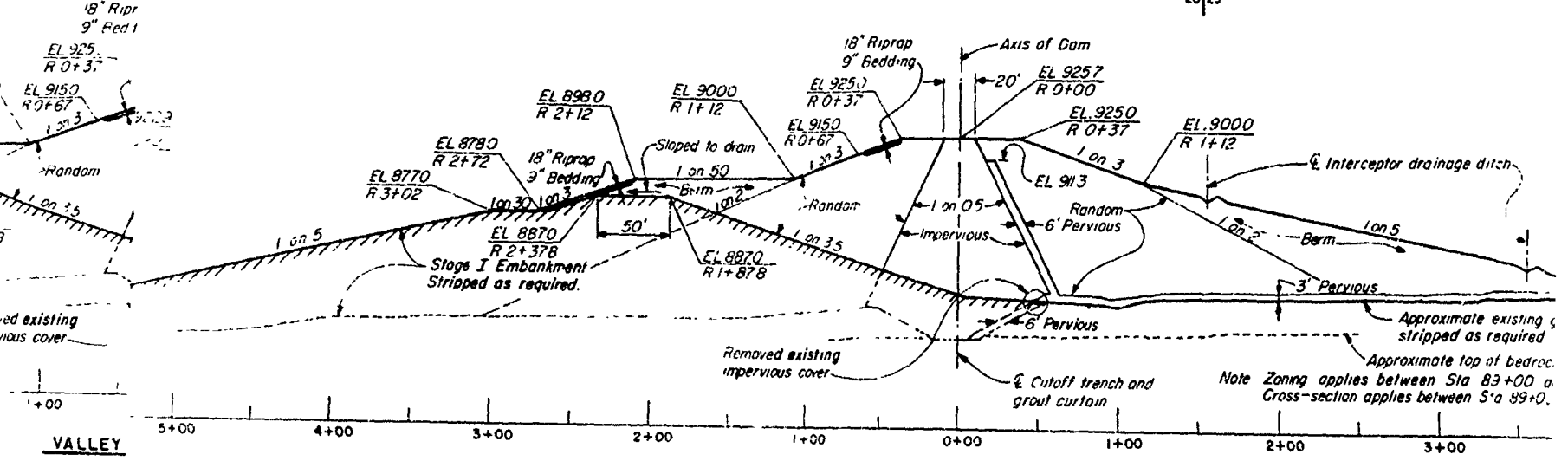
Constructed prior to diversion and closure

Removed existing

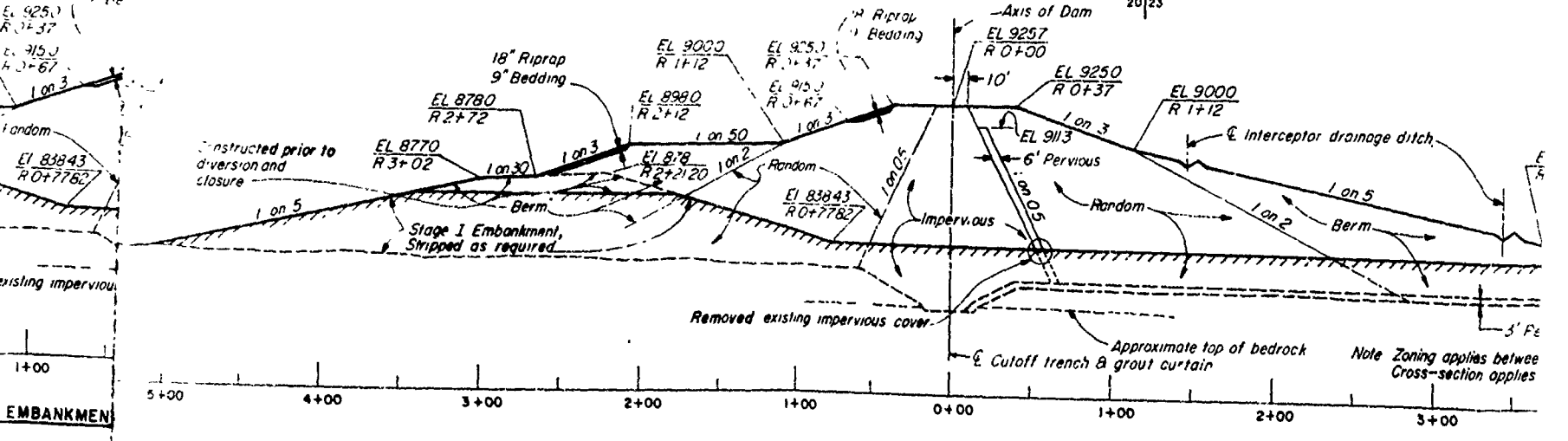
EM



RIGHT ABUTMENT SECTION - STA. 88+00

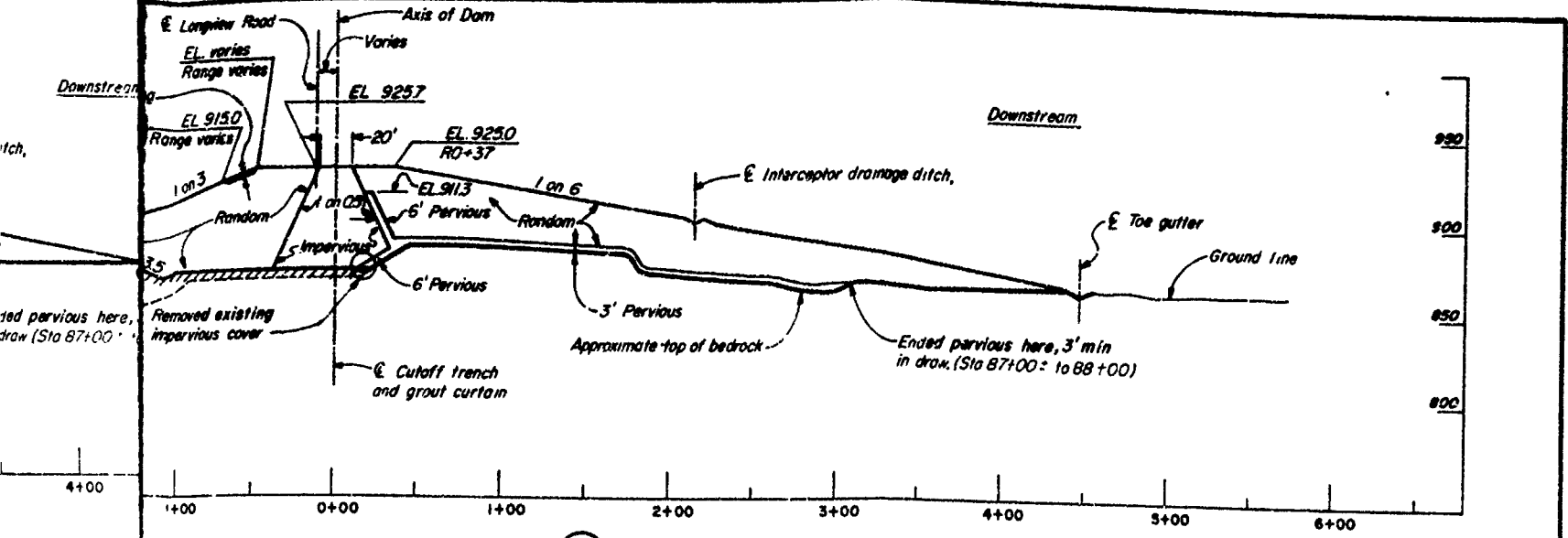


VALLEY SECTION - STA. 90+00

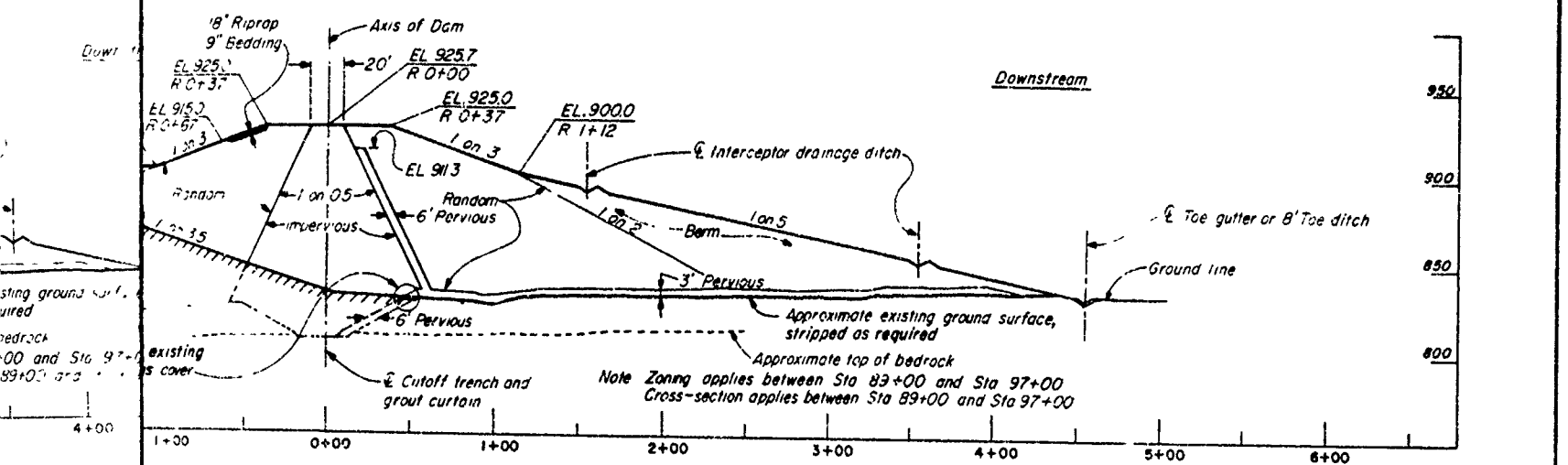


EMBANKMENT SECTION - STA. 97+00

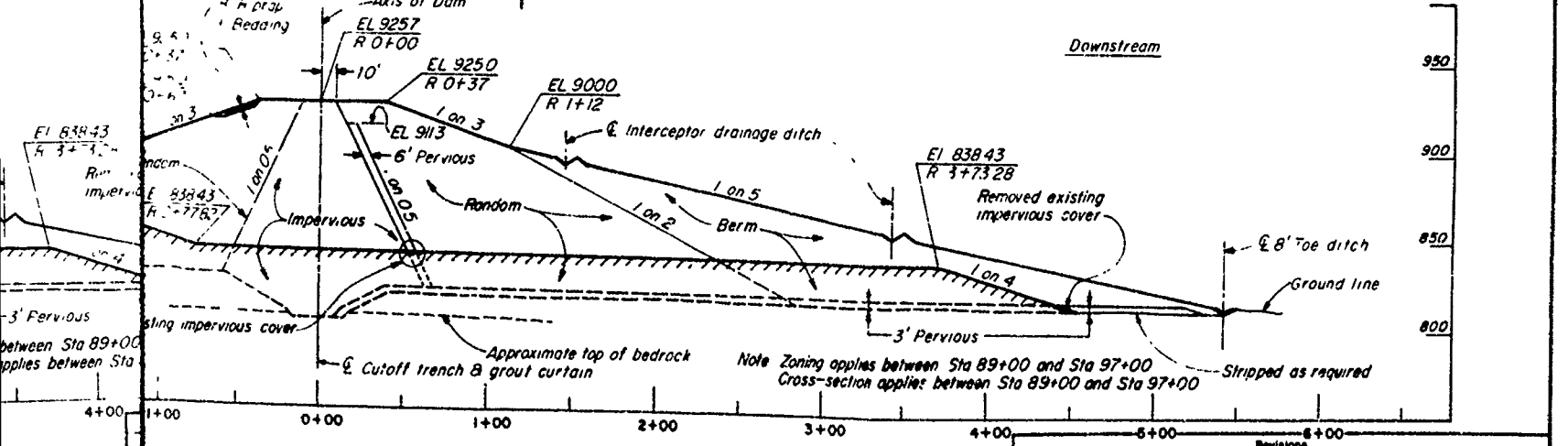
Notes 1 Hatched area represents approx. Stage I embankment  
2 Top, oiled and seeded all upstream embankment slopes, EL 898 and all downstream embankment slopes, areas having slope protection as shown on plan sections



RIGHT ABUTMENT SECTION - STA. 88+00  
A  
20/23




VALLEY SECTION - STA. 90+00  
B  
20/23

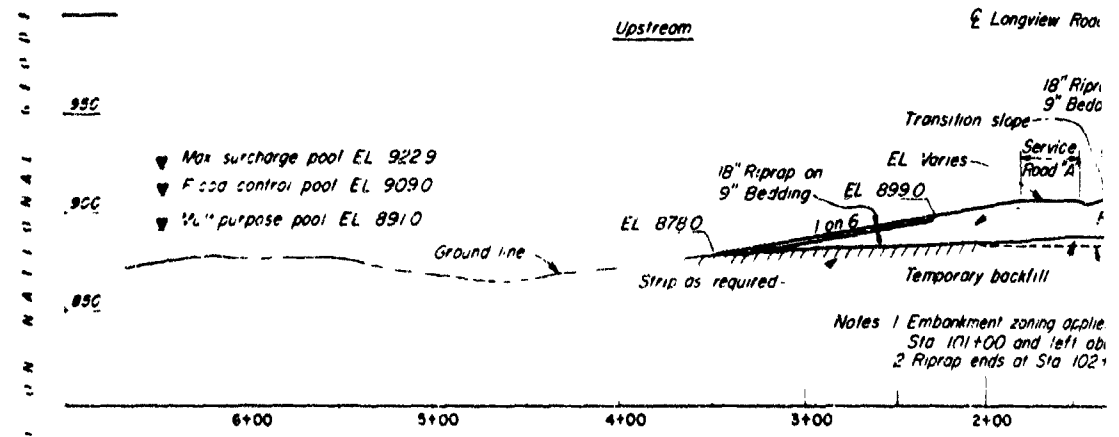
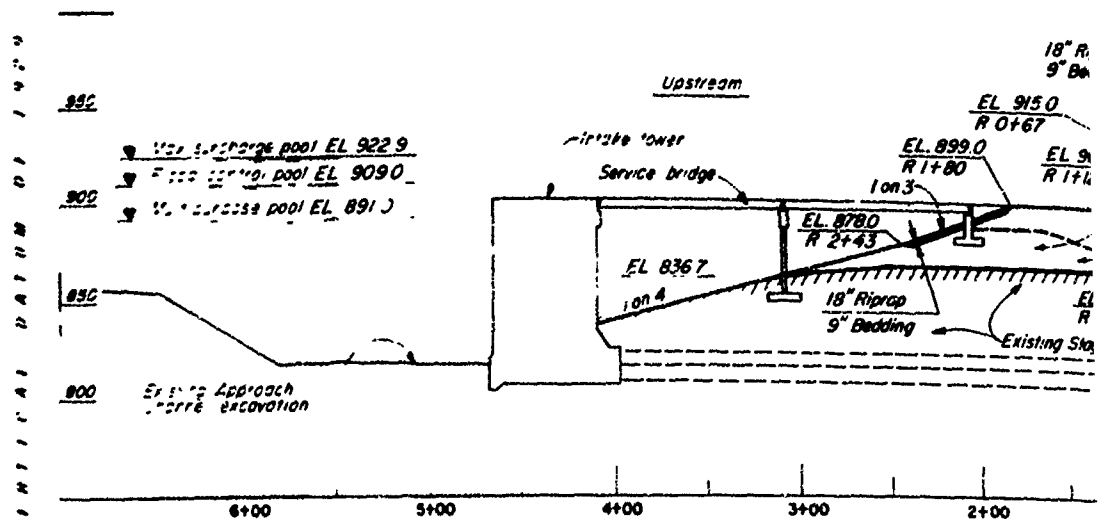


EMBANKMENT SECTION - STA. 97+00  
C  
20/23

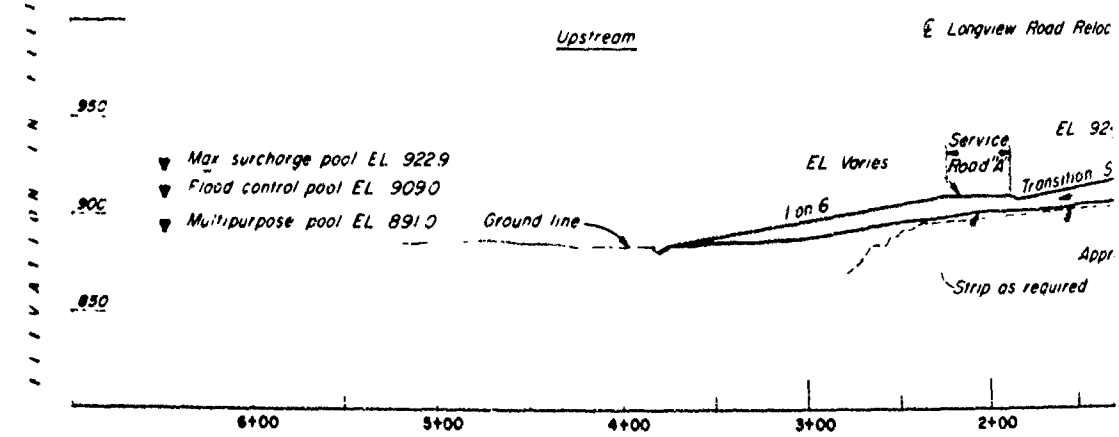
Notes: 1 Hatched area represents approx Stage I embankment  
2 Riprapped and seeded all upstream embankment slopes above EL 898 and all downstream embankment slopes except areas having slope protection as shown on plan and sections

Revisions			
Symbol	Descriptions	Date	Approved

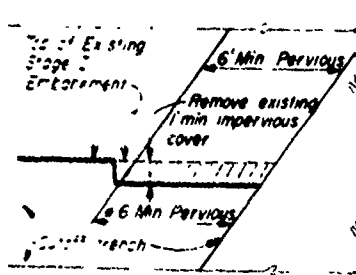
U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI		
Designed by R G D	 LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT  TYPICAL EMBANKMENT SECTIONS	
Drawn by S E S		
Checked by R G D / W B E		
Submitted by		
Scale AS SHOWN	Sheet Number 2 of 4	Draw No RBL-3-1423



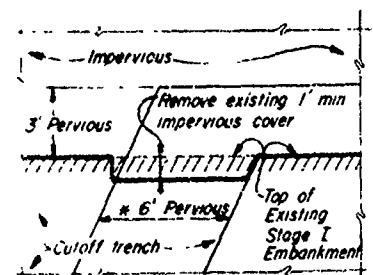
LEFT ABUTMENT SECTION - STA 102+00 (AXI)



LEFT ABUTMENT SECTION - STA 103+00 (AXI)

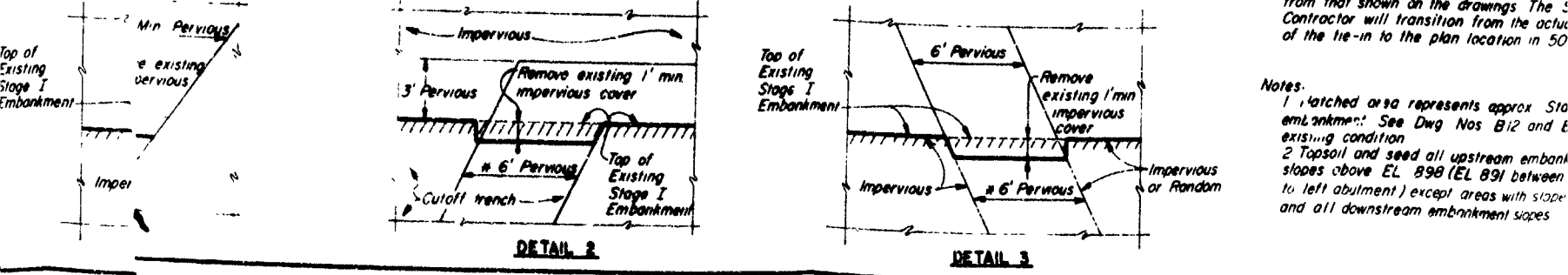
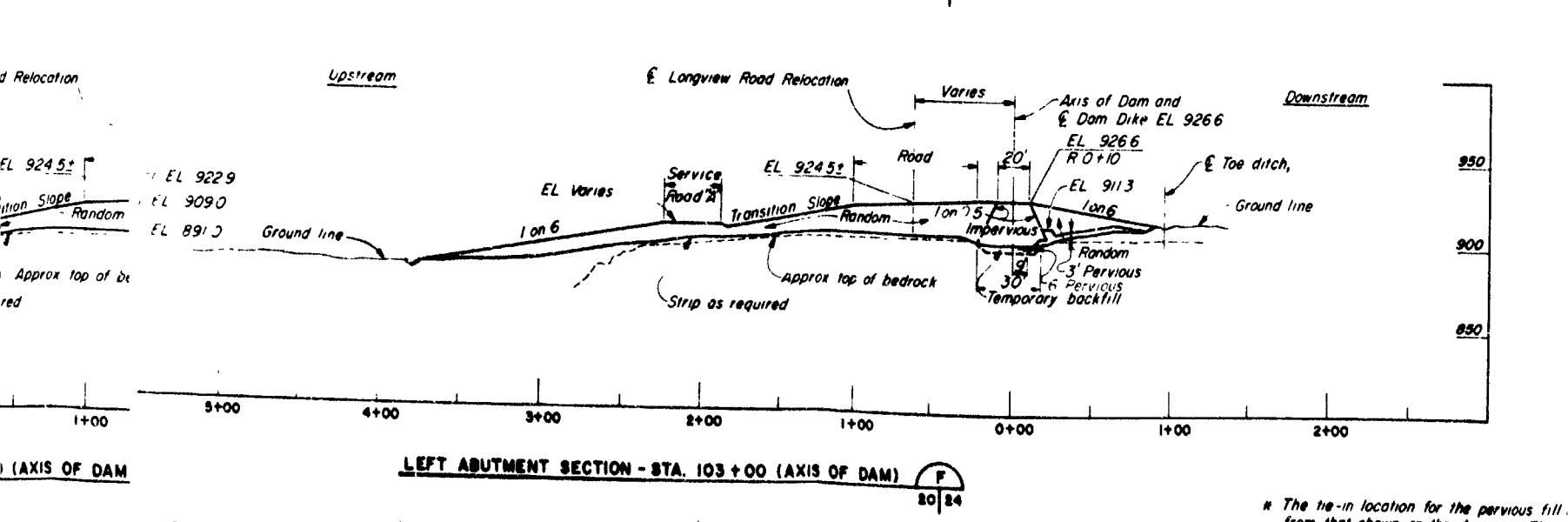
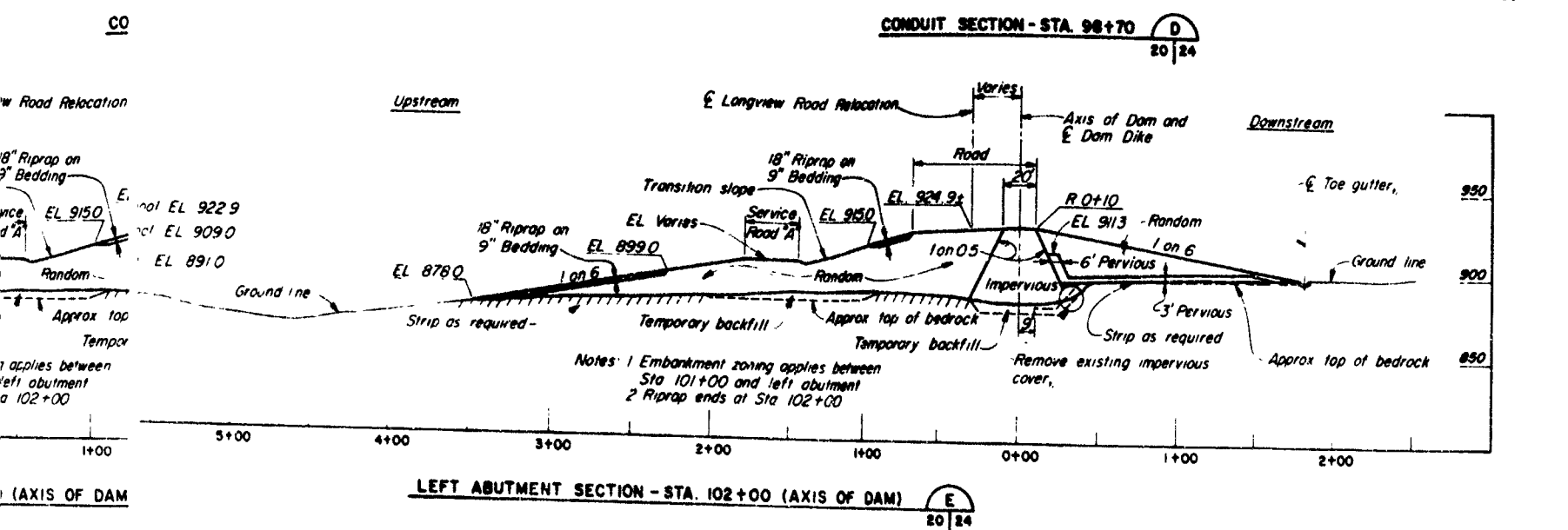
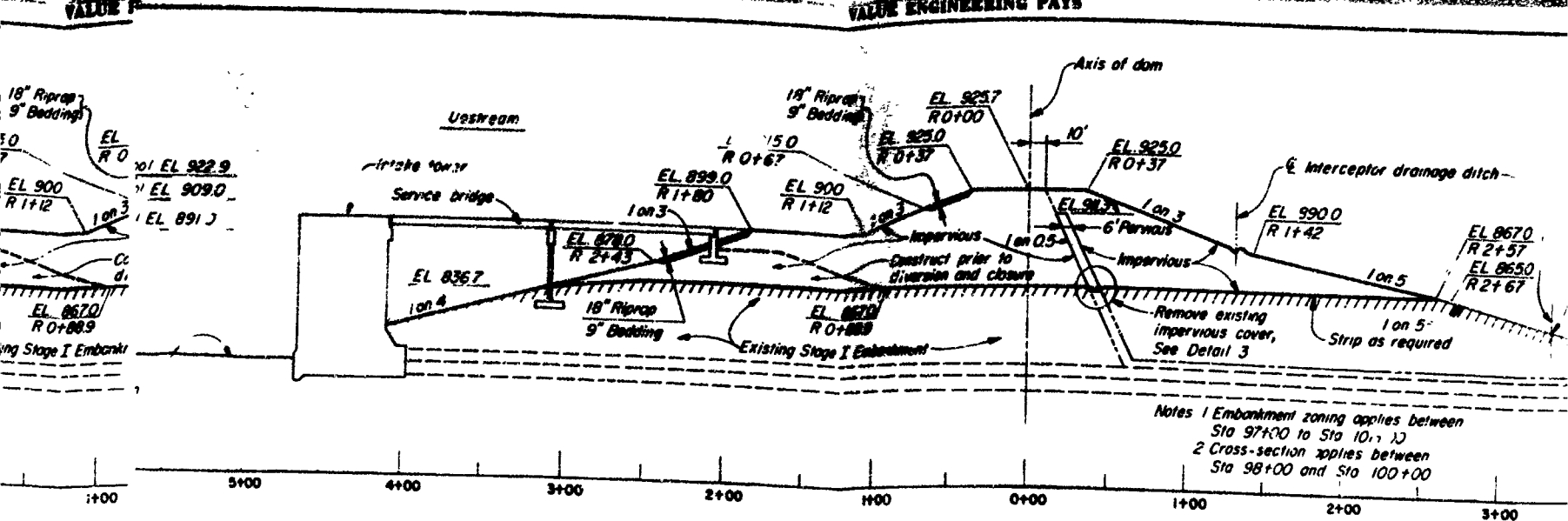


DETAIL 1



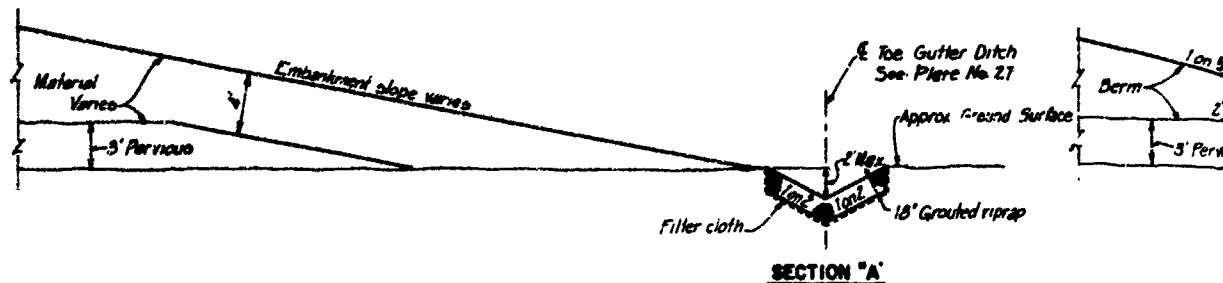
DETAIL 2

Top of Existing Stage I Embankment







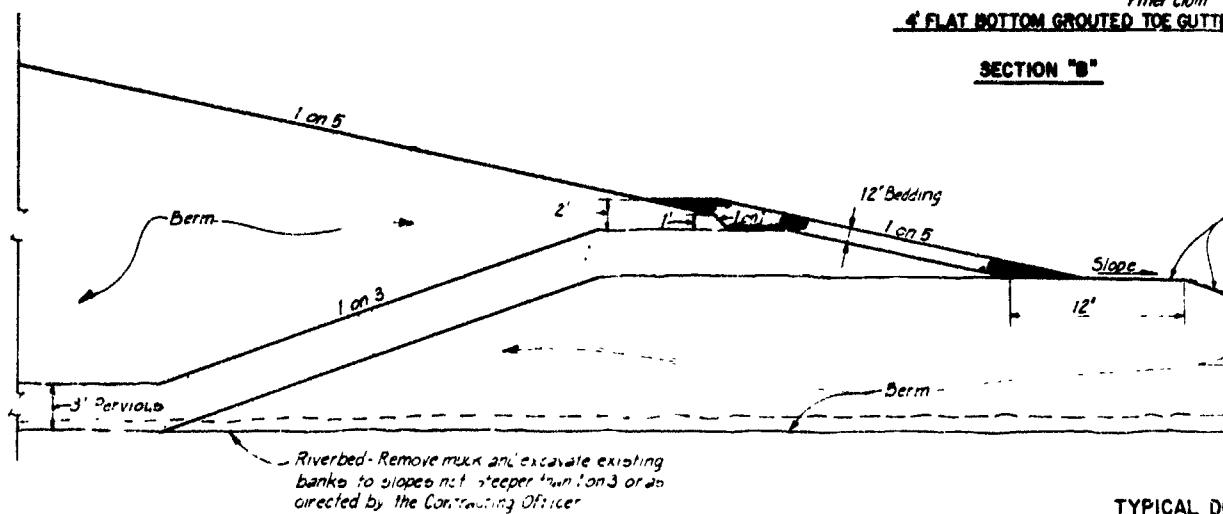


**TYPICAL DOWNSTREAM TOE DETAIL**

RIGHT ABUTMENT TO STA. 87+00  
STA. 88+00 TO STA. 91+00  
STA. 97+00 TO LEFT ABUTMENT  
Scale 1"=5'-0"

**4' FLAT BOTTOM GROUTED TOE GUTTER**

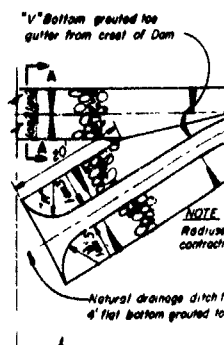
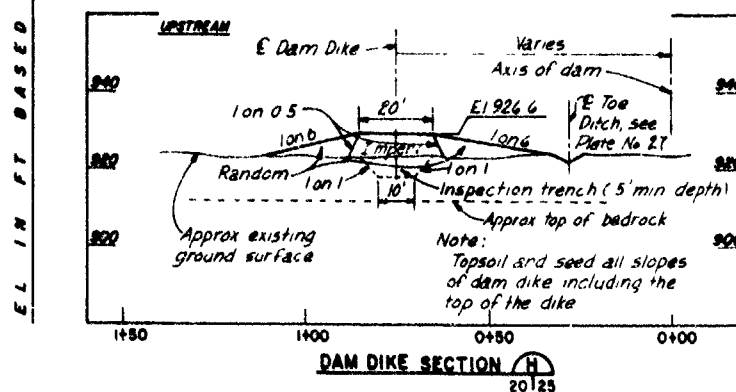
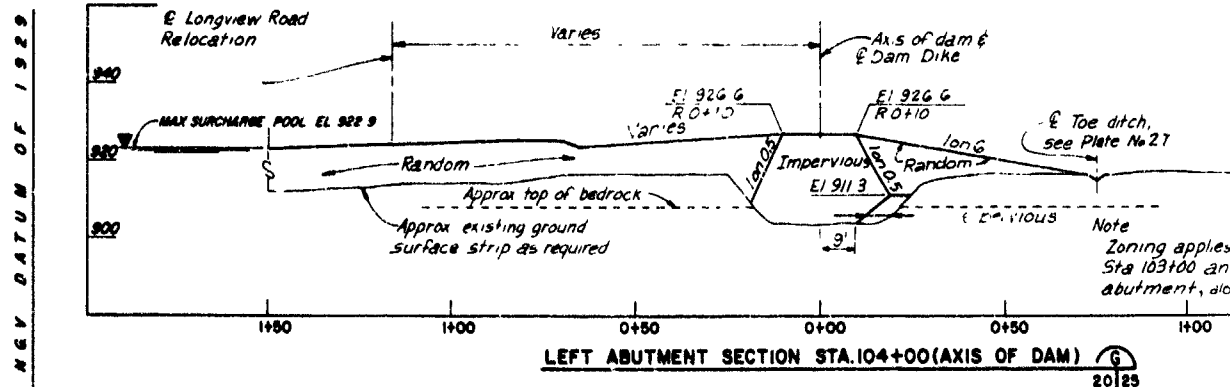
**SECTION "B"**



**TYPICAL DOWNSTREAM TOE DETAIL**

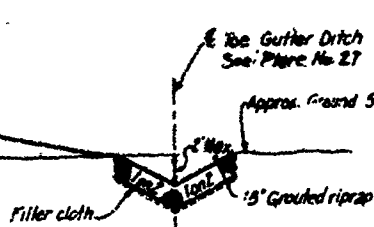
**THROUGH SECTION**

STA. 104+00

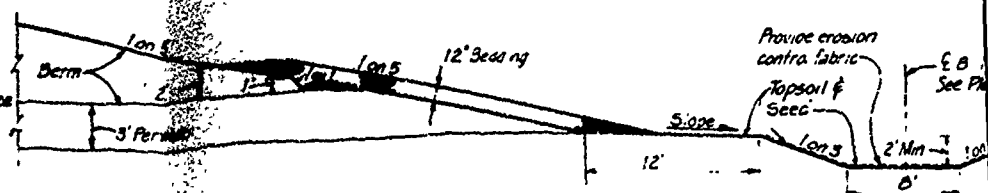


NOTE: THE REVISED GUTTER SHALL BE CONSTRUCTED AS SHOWN IN THE REVISION DATED 1983. THE FLOWLINE OF THE DITCH AT STA. 104+00 SHALL BE AT THAT ELEVATION SINCE THE GROUND ELEVATION AT THAT POINT IS 911.3. THE 6'-FOOT FLAT BOTTOM DITCH BETWEEN STA. 103+00 AND STA. 104+00 SHALL ACCOMMODATE RAISING THE DITCH GRADE EXACT OR APPROXIMATELY 1' IN THE FIELD BUT ARE TO BE NO FLATTER THAN SHOWN.

# VALUE ENGINEERING PAYS



SECTION "A"



SECTION "C"

## TYPICAL DOWNSTREAM TOE DETAIL

STA 91+00 TO STA 97+00  
Scale: "1"=5'0"



SECTION "B"

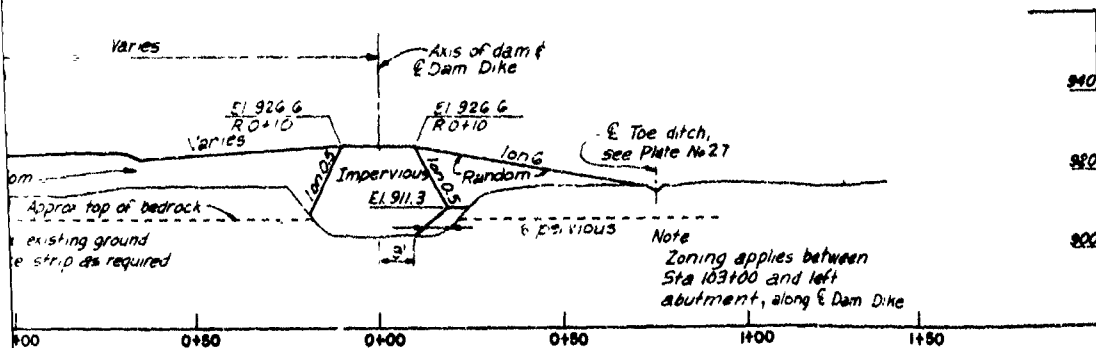
## 4' FLAT BOTTOM GROUTED TOE GUTTER

## TYPICAL DOWNSTREAM TOE DETAIL

### THRU RIVER CHANNEL

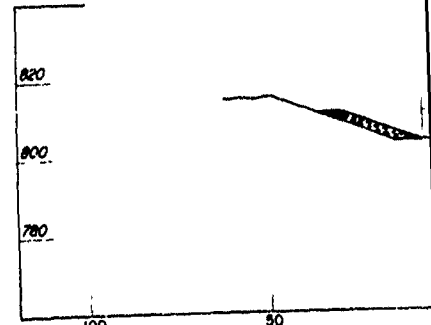
STA 94+00 TO STA 95+50  
Scale: "1"=5'0"

to rock and excavate existing  
bank steeper than 1 on 3 or as  
Contracting Officer

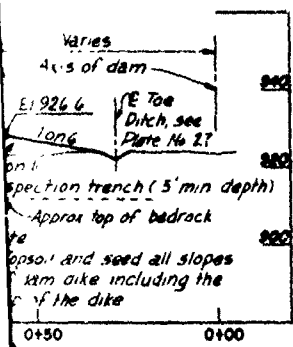


LEFT ABUTMENT SECTION STA. 104+00 (AXIS OF DAM)

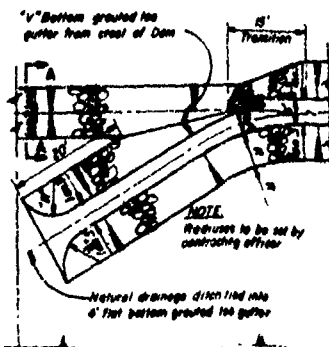
20125



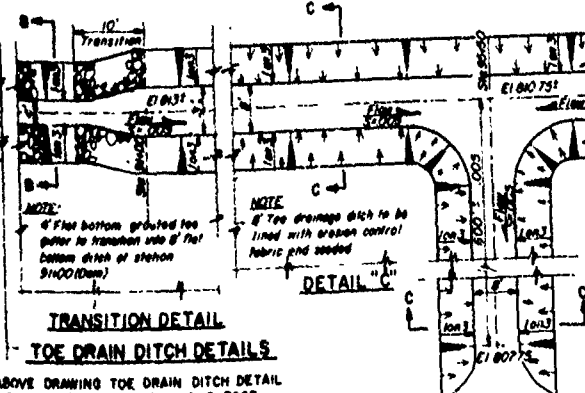
SEC



DETAIL "A"

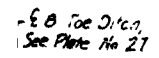


DETAIL "B"



TRANSITION DETAIL  
TOE DRAIN DITCH DETAILS

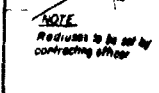
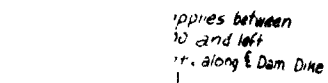
NOTE:  
THE REVISED GUTTER SHALL BE CONSTRUCTED AS SHOWN ON THE ABOVE DRAWING TOE DRAIN DITCH DETAIL  
DATED 1993. THE FLOWLINE OF THE DITCH AT ST. 91+00 SHALL BE REVISED TO MAINTAIN A 2-FOOT  
DEPTH SINCE THE GROUND ELEVATION AT THAT POINT IS APPROXIMATELY ELEVATION 819.23. THE GRADE  
OF THE 8-FOOT FLAT BOTTOM DITCH BETWEEN STA 91+00 AND STA 95+50 WILL HAVE TO BE STEEPENED TO  
ACCOMMODATE RAISING THE DITCH GRADE. EXACT GRADES OF THE 8-FOOT FLAT BOTTOM DITCH ARE TO BE DETERMINED  
IN THE FIELD BUT ARE TO BE NO FLATTER THAN SHOWN ON THE ABOVE DRAWING. ALSO SEE PLATE NO 27



"C"  
L  
  
Green  
[redacted]  
xited  
'p  
cloth  
GUTTB



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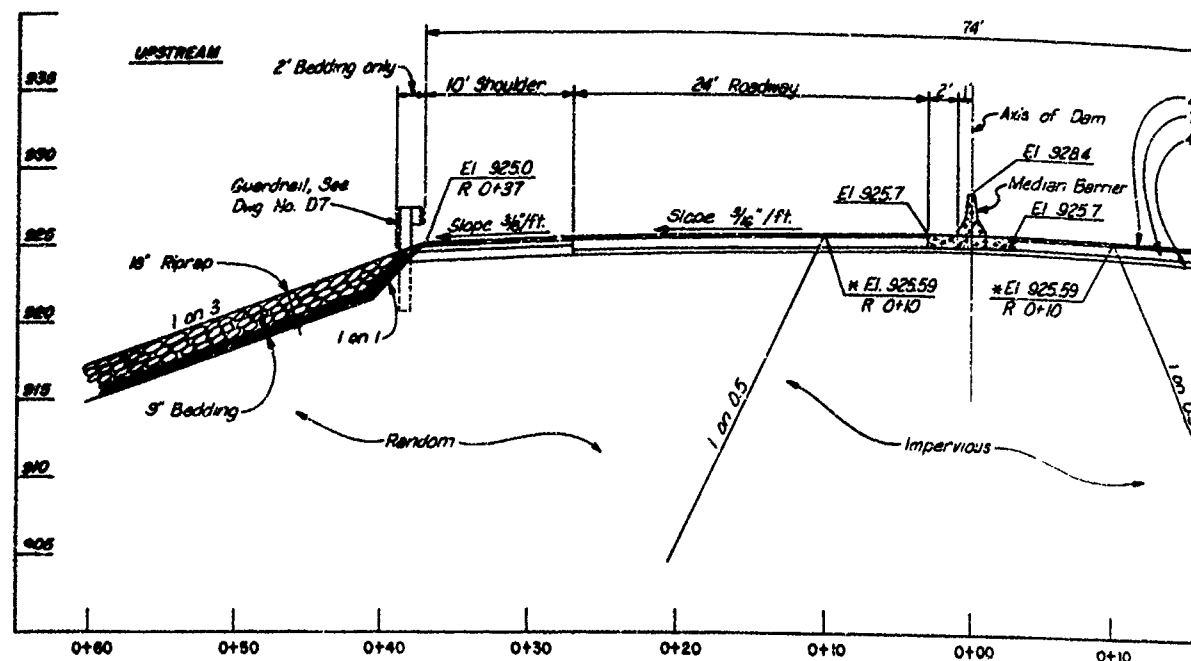


AS SHOWN ON THE ABOVE DRAWINGS THE DRAIN DITCH DETAIL AT STA 81+00 SHALL BE REVISED TO MAINTAIN A 2-FOOT DEPTH. THIS IS APPROXIMATELY ELEVATION 819.27. THE GRADE BETWEEN STA 81+00 AND STA 86+90 WILL HAVE TO BE STEEPENED TO MAINTAIN THE 8-FOOT FLAT BOTTOM DITCH AREA TO BE DETERMINED SHOWN ON THE ABOVE DRAWING. ALSO SEE PLATE NO 27

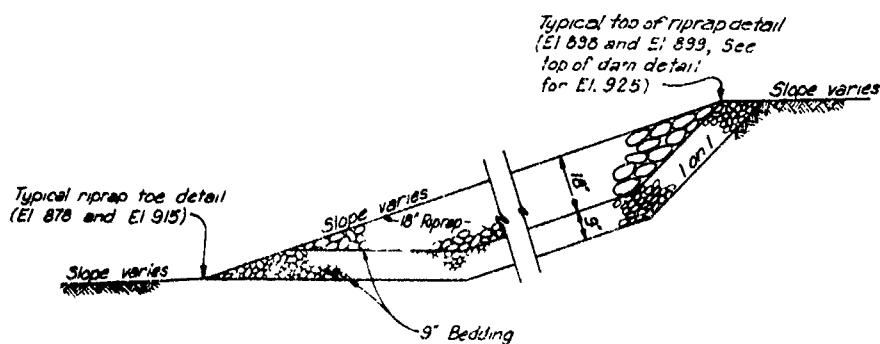
U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Scale	AS SHOWN	Sheet Number	Plot Scale
Date			Design Firm
Dwg No.		4 OF 4	File No. RDL-3-1425

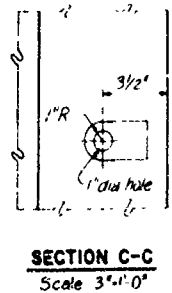
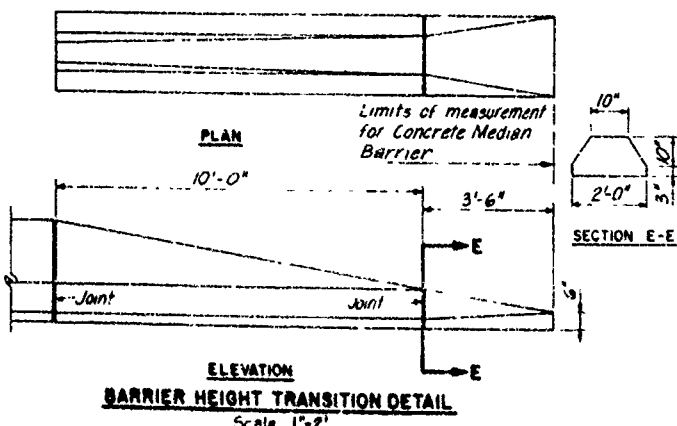
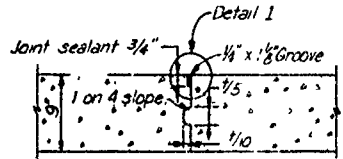
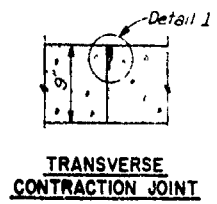
EL. IN FT. BASED ON M.S.V. DATUM OF 1922

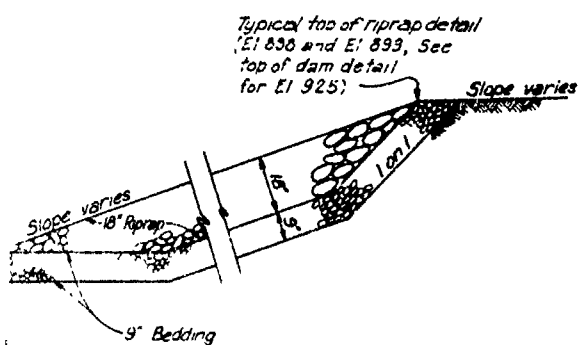
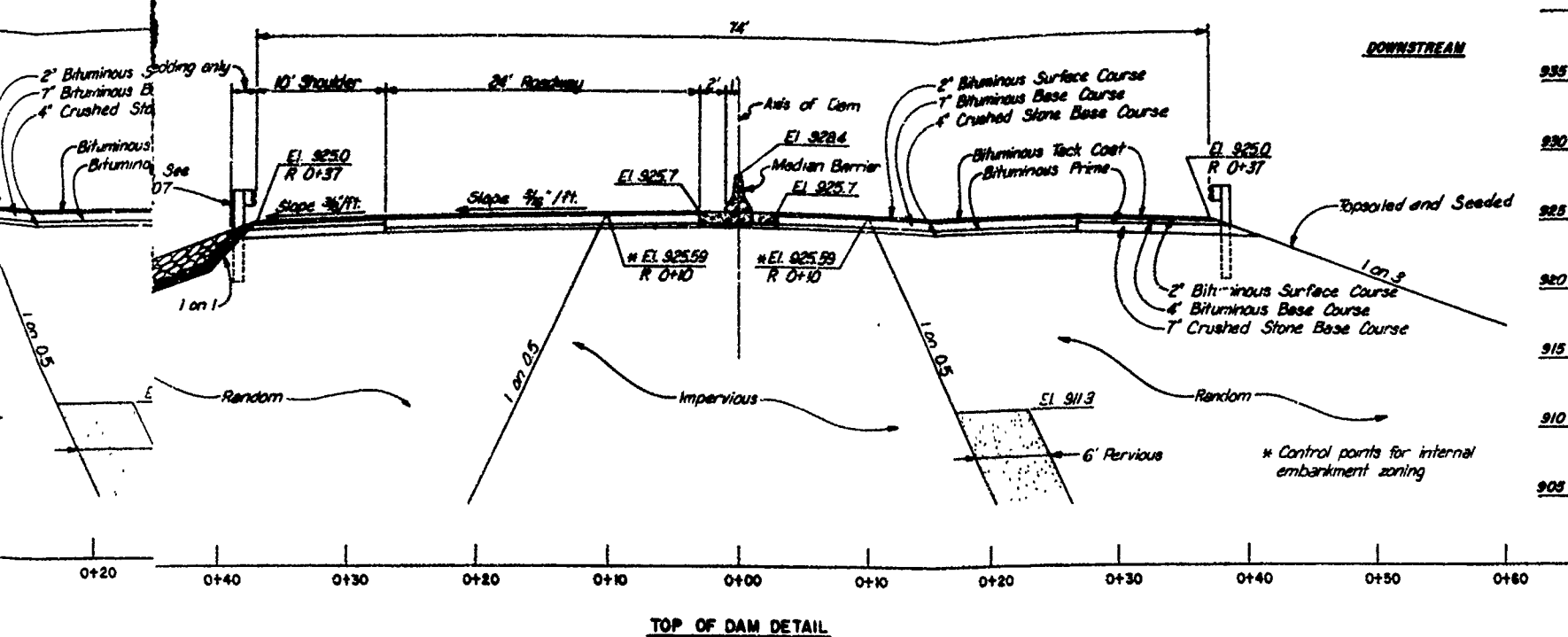


TOP OF DAM DETAIL

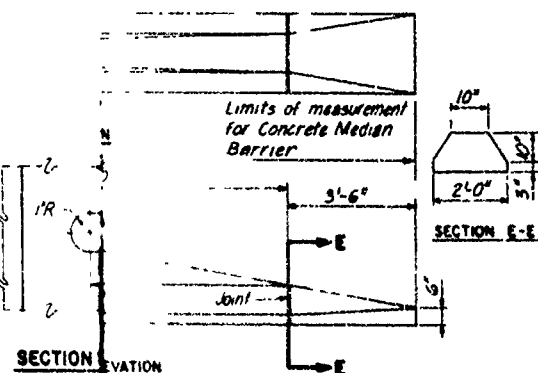
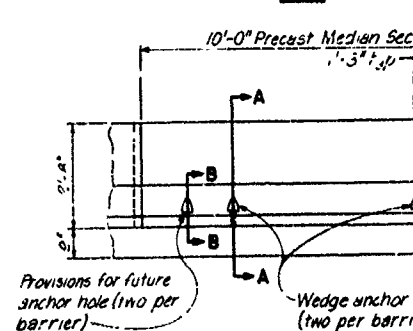
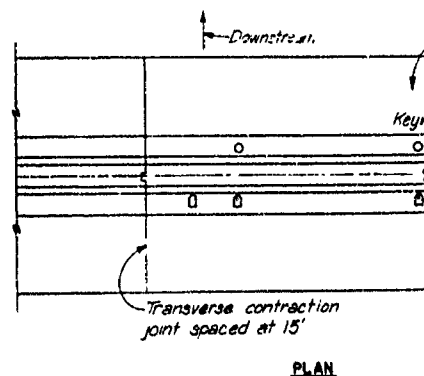
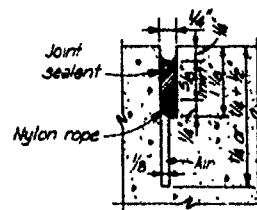
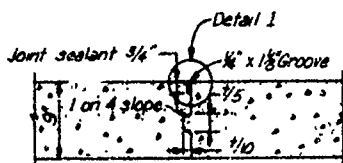


SLOPE PROTECTION DETAIL  
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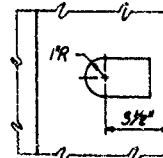
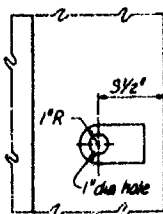




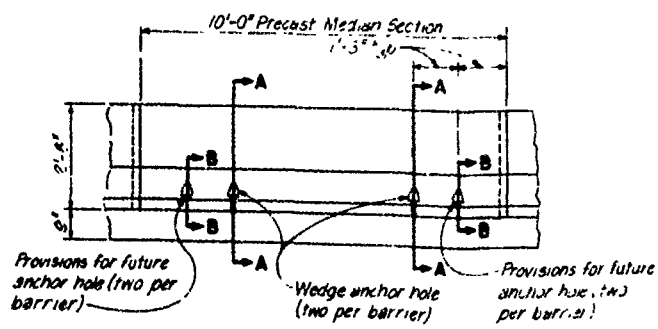
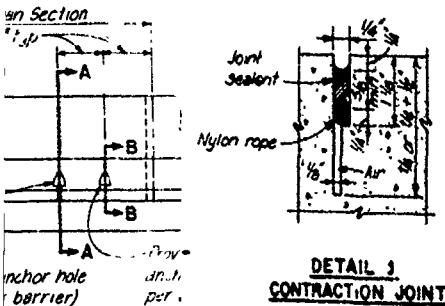
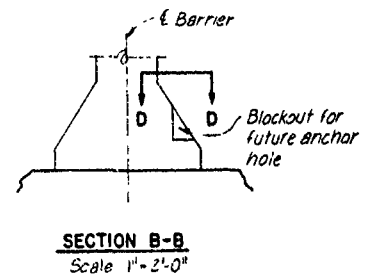
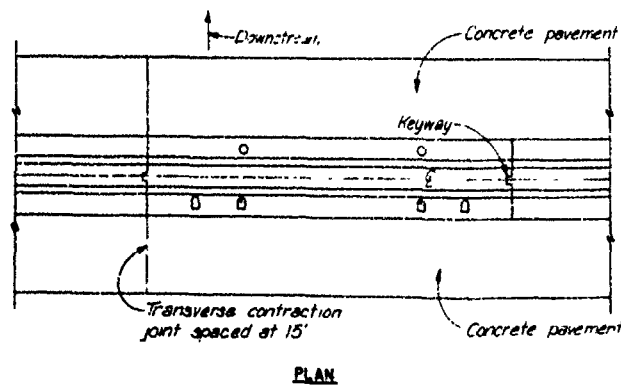
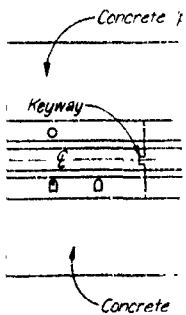
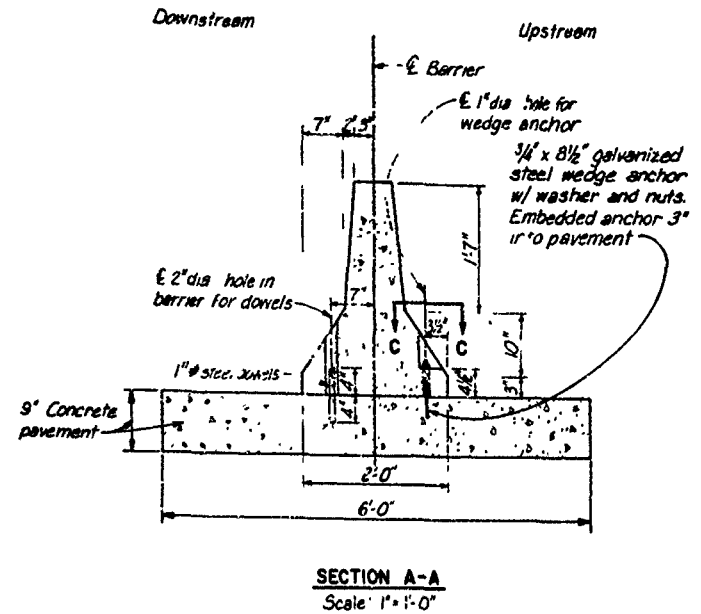
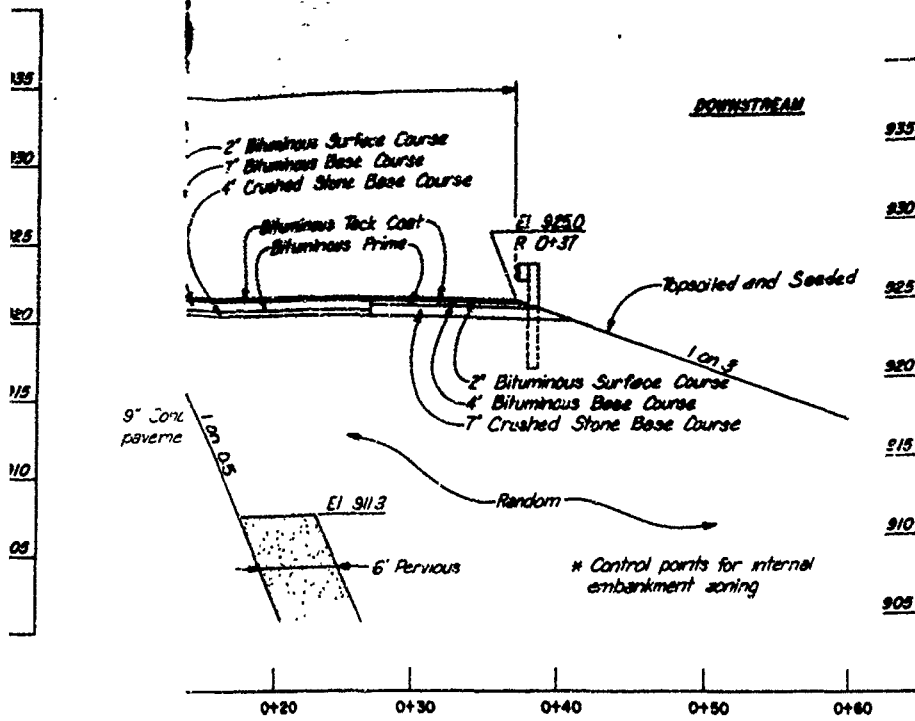
**TRANSVERSE CONTRACTION JOINT**



**SECTION C-C**  
Scale: 3" = 1'-0"

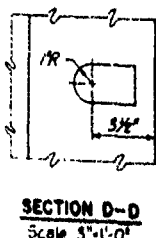


**SECTION D-D**  
Scale: 3" = 1'-0"



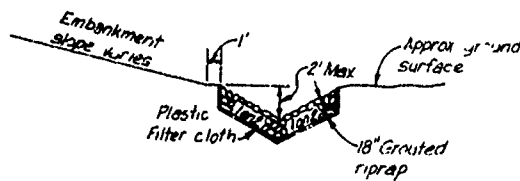
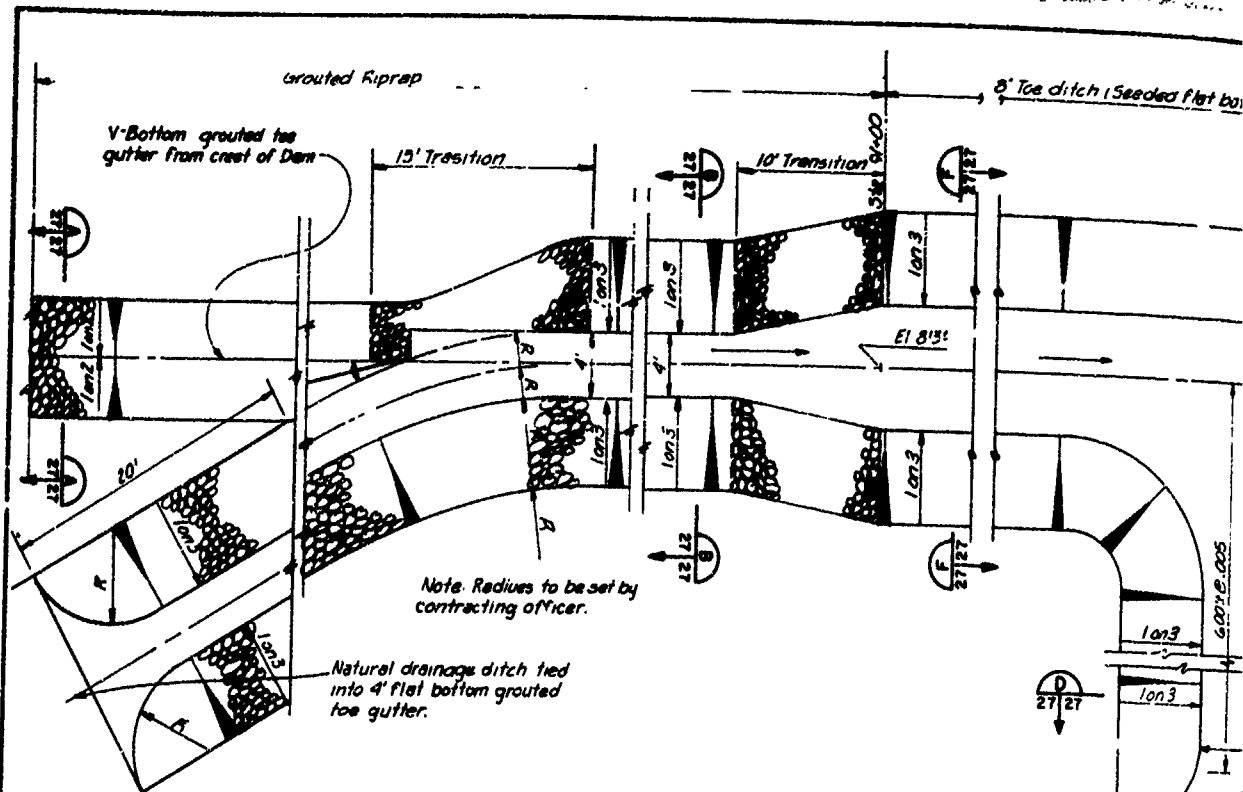
- NOTES**
1. Provided reinforcing as required for handling and to meet Missouri Highway and Transportation Commission Concrete Median Barrier Drawing 617 OOP
  2. For keyway and other details not shown, see Drawing 617 OOP, Concrete Median Barrier, Missouri Highway Transportation Commission
  3. Anchors for transition sections similar to those shown for other barrier precast sections.

AN BARRIER

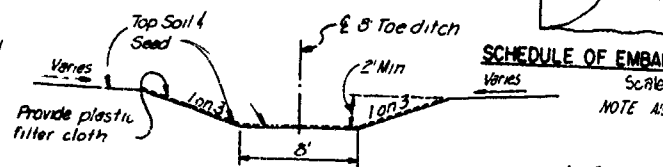


**ELEVATION  
UPSTREAM SIDE OF MEDIAN BARRIER**  
Scale 1" = 2'-0"

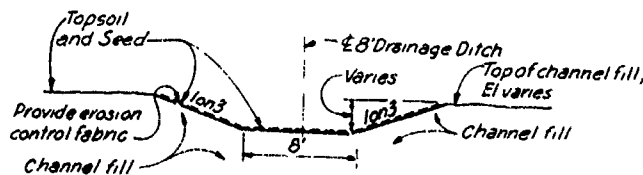
Revisions			
Symbol	Descriptions	Date	Approved
U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	R G D	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by	S E S	TOP OF DAM AND SLOPE PROTECTION DETAILS	
Checked by	R G D / W B E	Scale AS SHOWN	Sheet number
Submitted by	R G D	Date	File No
			RBL-3-1426



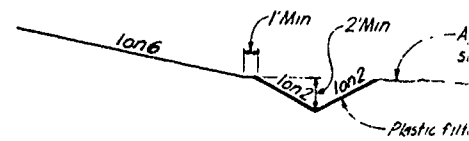
**TOE GUTTER DITCH SECTION A**  
Scale 1"=5'-0"  
FROM RIGHT ABUTMENT THRU STA 90+85  
AND STA 99+10



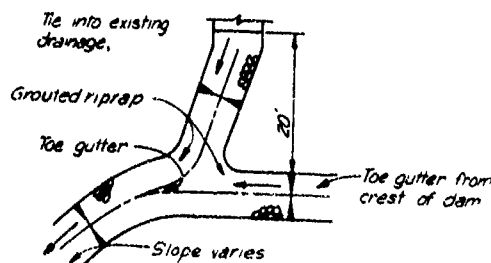
**8' TOE DITCH SECTION F**  
Scale 1"=5'-0"  
STA 91+00 THRU STA 91+00



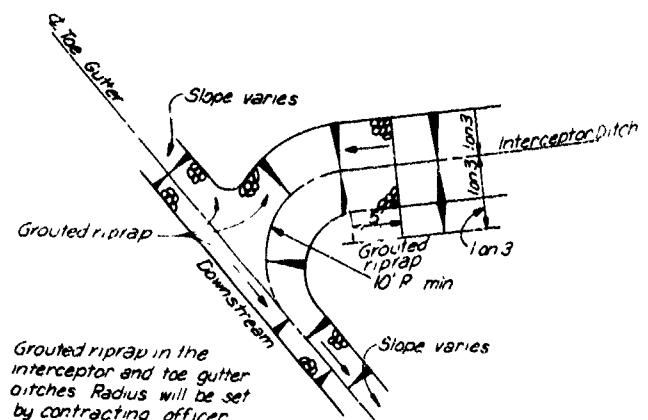
**8' DRAINAGE DITCH SECTION D**  
DOWNSTREAM OF SERVICE ROAD D  
Scale 1"=5'-0"



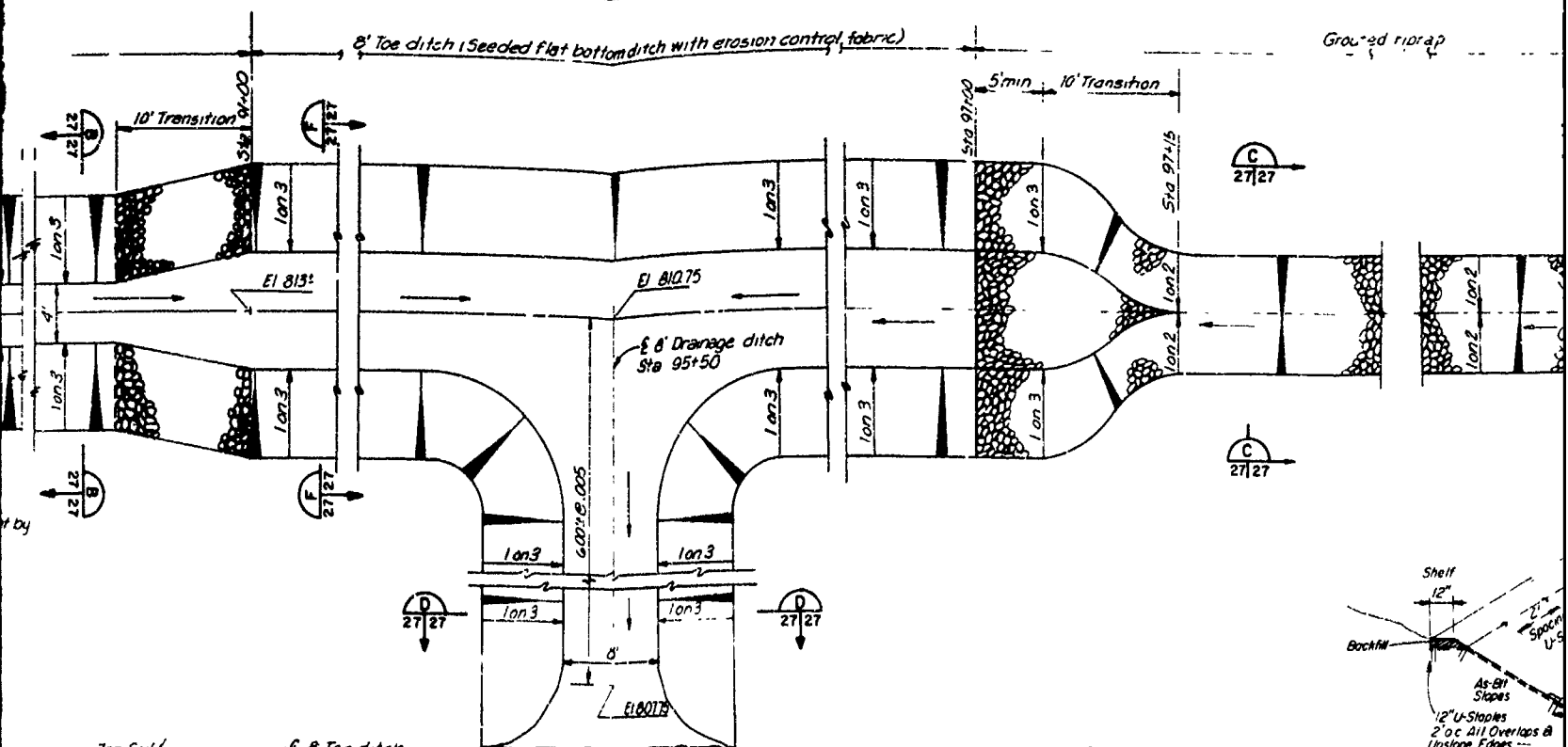
**TOE DITCH DETAIL SECTION E**  
DAM STA 103+00 THRU STA 107+50  
Scale 1"=5'-0"



**DETAIL A**  
TOE GUTTER TIE IN DETAIL RIGHT ABUTMENT  
Not to Scale



**TYPICAL INTERCEPTOR DITCH**  
TOE GUTTER CONNECTION DETAIL  
Not to Scale



**SCHEDULE OF EMBANKMENT DRAINAGE**

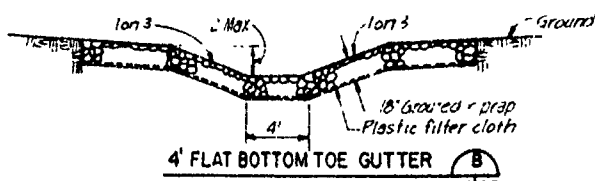
Scale 1"=5'-0"

NOTE: Also see TOE DRAIN DITCH DETAIL

**8' TOE DITCH SECTION**

Scale 1"=5'-0"

STA 91+00 THRU STA 94+00



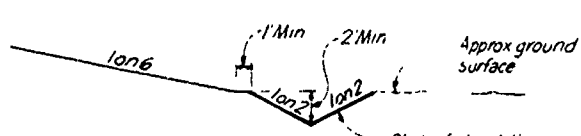
**4' FLAT BOTTOM TOE GUTTER**

Scale 1"=5'-0"

**TOE GUTTER DITCH SECTION**

Scale 1"=5'-0"

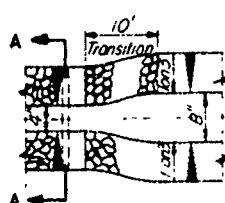
DAM STA 97+15 THRU STA 99+00



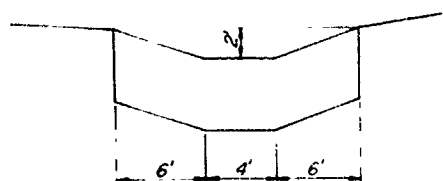
**TOE DITCH DETAIL SECTION**

DAM STA 103+00 THRU STA 107+50

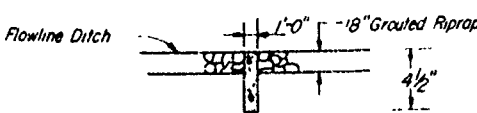
Scale 1"=5'-0"



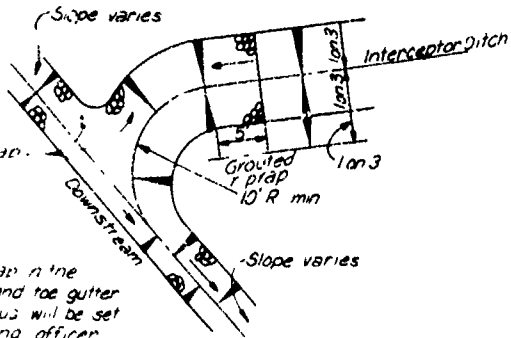
**SECTION AA CUTOFF WALL**



**CUTOFF WALL**

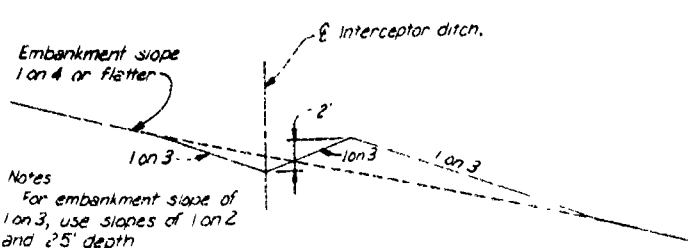


**SECTION AA CUTOFF WALL**



**TYPICAL INTERCEPTOR DITCH TOE GUTTER CONNECTION DETAIL**

Not to Scale



**TYPICAL INTERCEPTOR DITCH DETAIL**

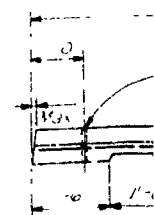
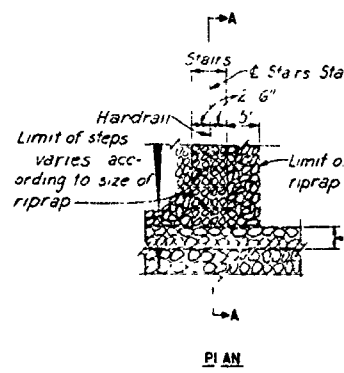
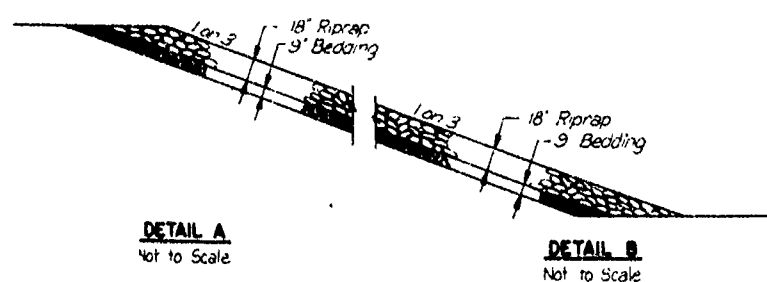
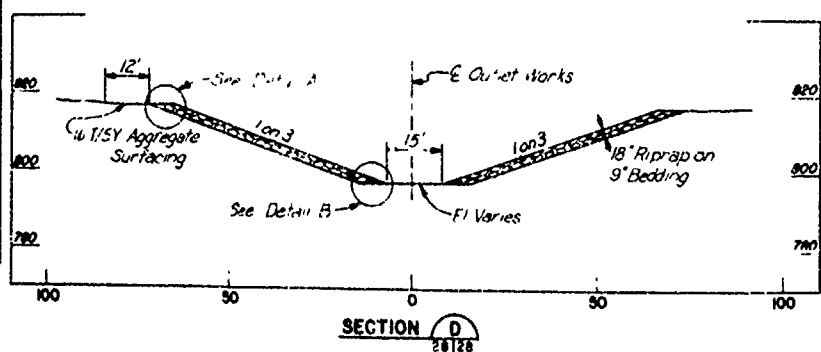
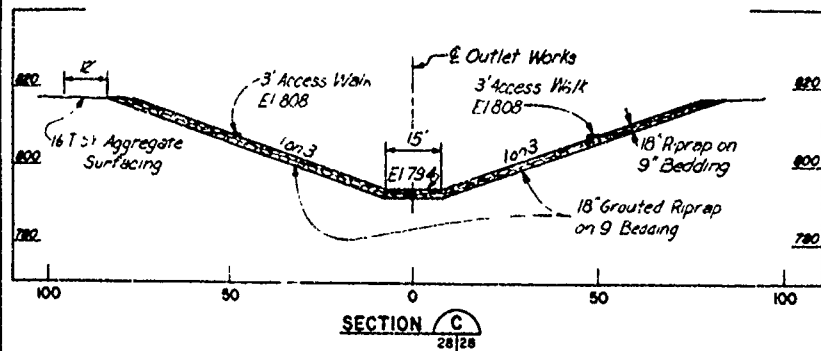
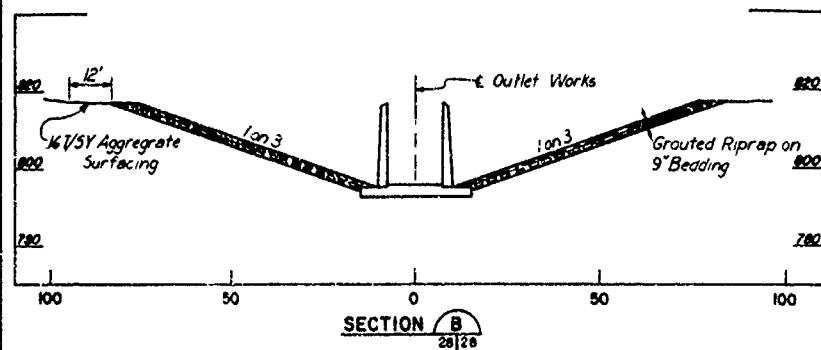
Notes  
For embankment slope of 1 on 3, use slopes of 1 on 2 and 2'5" depth  
Sta 94+00 R+37.6 width of ditch 10' and depth 2'5" and 18" grouted riprap

Symbol	
Designation	
Notes	
Remarks	
Drawn by	
Checked by	
Approved by	

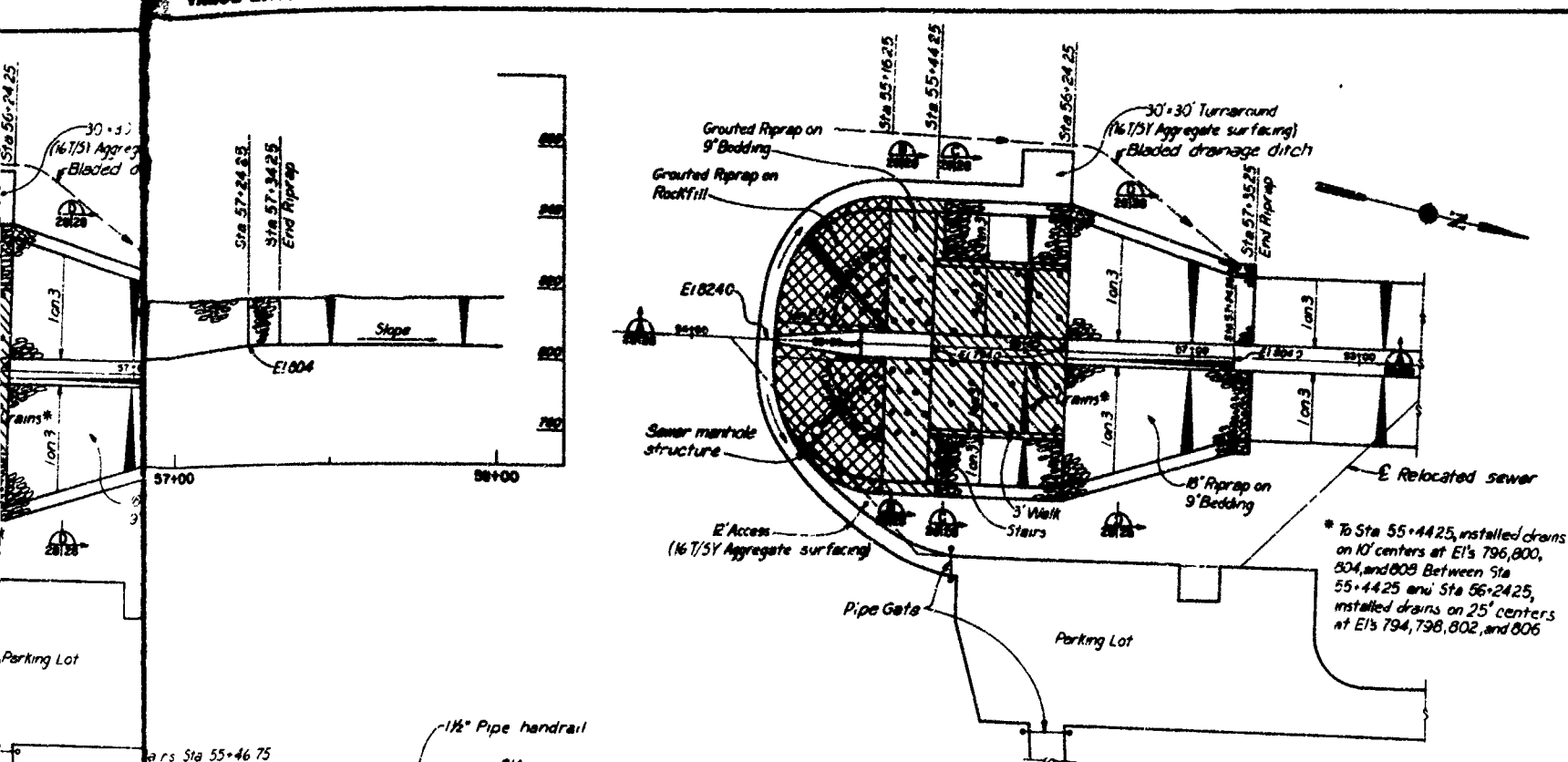




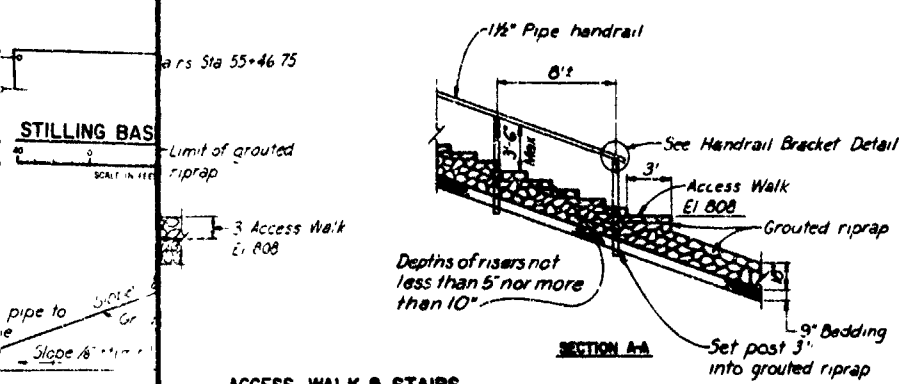
Profile A of the proposed 12' access channel. The profile shows a cross-section of the channel with a 1 on 4 slope on the left and a 1 on 10 slope on the right. Key features include a toe gutter, 12' access, and a 12' access. The profile is labeled with stationing from 54+00 to 57+00 and elevation from 780 to 820. Key points include Sta 54+31.00, Sta 55+46.25, Sta 55+46.75, Sta 56+24.25, and Sta 56+24.25. The profile is labeled 'PROFILE A' and '28/28'.



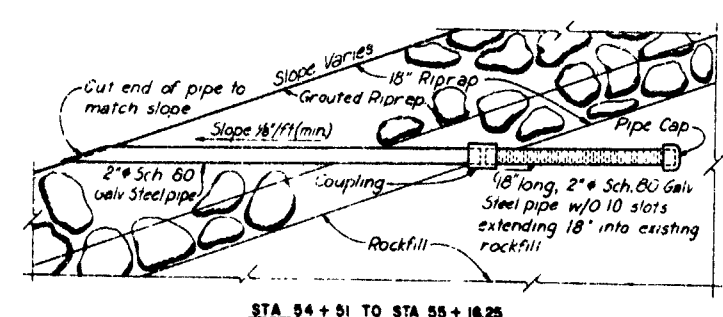




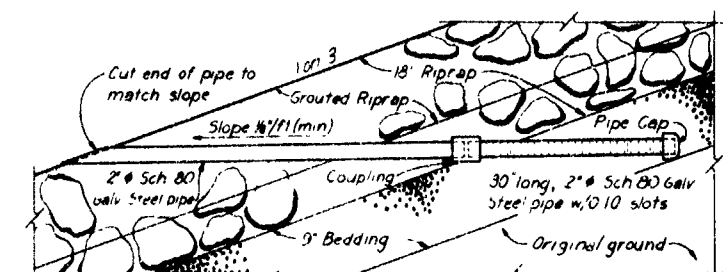
**STILLING BASIN PLAN**  
SCALE IN FEET



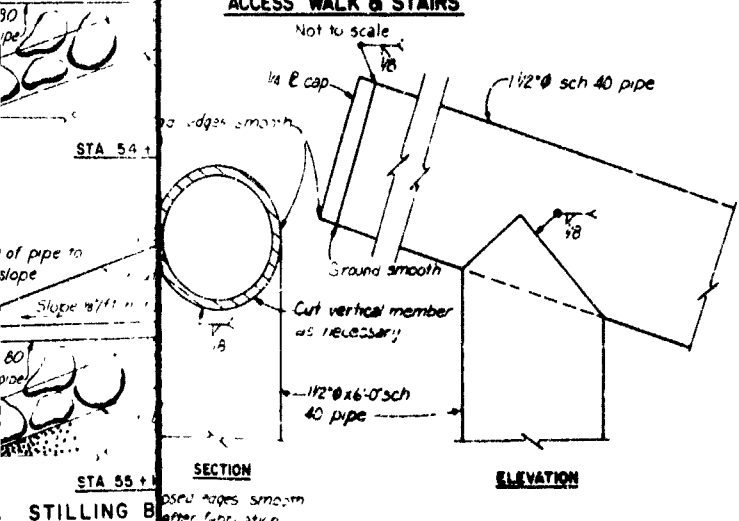
**ACCESS WALK & STAIRS**  
Not to scale



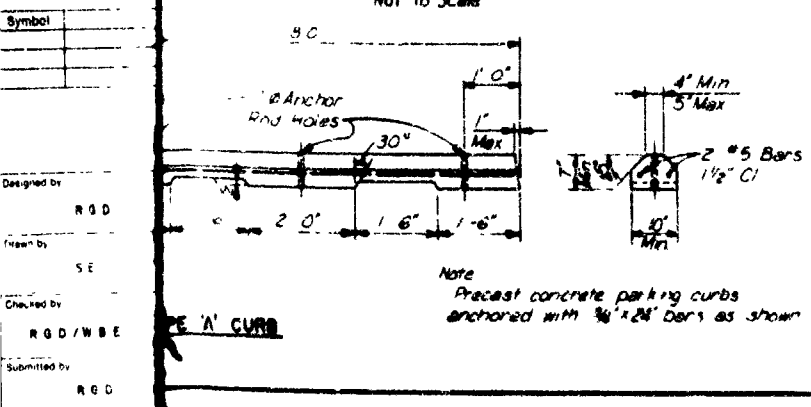
**STA 54 + 51 TO STA 55 + 16.25**



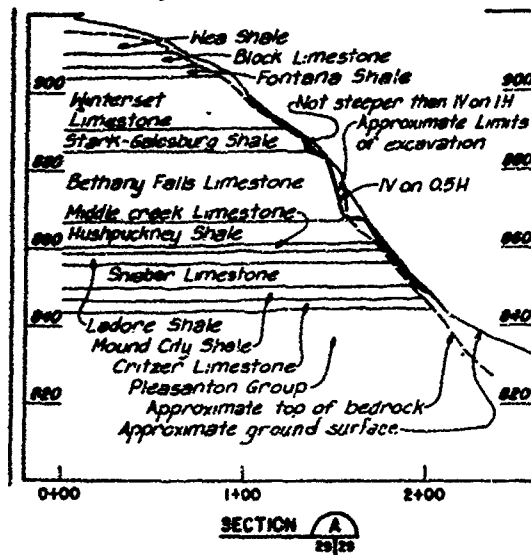
**STA 55 + 16.25 TO STA 56 + 24.25**  
**STILLING BASIN OUTLET DRAIN**  
Not to scale



**HANDRAIL DETAILS**  
Not to Scale



Revisions			
Symbol	Descriptions	Date	Approved
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	RGD	<b>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</b>  <b>STILLING BASIN: PLAN, PROFILE, SECTIONS AND DETAILS</b>	
Drawn by	SES		
Checked by	RGD/WBE		
Submitted by	RGD		
Scale	AS SHOWN	Sheet number	
Date			
Drawn by			
File No	RDL-3-1428		



Constructed bench as necessary to excavate Shinarump Limestone

Constructed bench as necessary to excavate Bethany Falls

El. 835

Not steeper than IV on 2H

Excavated IV on 0.5H through Shinarump Limestone

APPROXIMATE ABUTMENT EX LIMITS

Cut slope in Bethany Falls Limestone

6 Dam

STAGE I EMBANKMENT

El. 887 R+37.9

El. 878.3 R+64

10x10

1 on 4

1 on 3.5

5' to 6'

as necessary  
Bethany Falls Limestone

Constructed bench as  
necessary to excavate  
Sniabar Limestone

Constructed bench as necessary  
to excavate Bethany Falls Limestone

Notes:

- ① Limits of Stage II excavation shown are approximate. Actual vertical and horizontal limits depend upon conditions encountered and were as directed by the Contracting Officer.
- ② Stage I rock surfaces received foundation cleanup or preparation as directed by the Contracting Officer and removal of temporary protective soil cover.
- ③ All areas within the Stage II excavation limits were excavated to depths as directed, which removed all highly organic material, loose rock and any other objectionable foundation materials as determined by the Contracting Officer.

APPROXIMATE STAGE II  
ABUTMENT EXCAVATION

El. 835

Not steeper  
than IV on 2H

Excavated IV on 0.5H  
through Sniabar  
Limestone

APPROXIMATE STAGE II  
ABUTMENT EXCAVATION  
LIMITS

Approximate embankment  
toe trace

Cut in slope  
in overburden

Cut slope in  
Bethany Falls  
Limestone

Slope in Wea  
and Block Members

Temporary protective soil  
cover removed

STAGE I EMBANKMENT

El. 887 R2+37.9

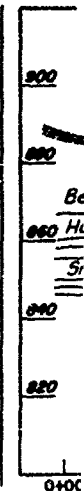
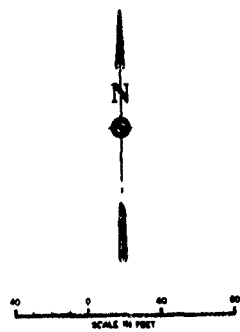
El. 878.3 R2+64

El. 877 R3+02

Cut slope in  
Winterset Limestone

Cut slope in  
Bethany Falls Limestone

Constructed bench as  
necessary to excavate  
Bethany Falls Limestone



EMBANKMENT

7.9

6.4

El. 877 R3

1 on 4

1 on 3.5

1 on 5

El. 898

El. 848

5.00

5.00

5.00

5.00

5.00

5.00

5.00

5.00

**VALUE ENGINEERING PAYS**

as necessary  
Bethany Falls Limestone

Approx. then

Bethany Falls Limestone  
Hushpuckney Shale  
Snider Limestone  
Critzler Limestone  
Pleasanton Group

Excavated IV on Q5H through Bethany Falls Limestone

Approximate embankment toe trace

**APPROXIMATE STAGE II EXCAVATION LIMITS**

Notes:

- ① Limits of Stage II excavation shown are approximate. Actual vertical and horizontal limits depend upon conditions encountered and were as directed by the Contracting Officer.
- ② Stage I rock surfaces received foundation cleanup or preparation as directed by the Contracting Officer and removal of temporary protective soil cover.
- ③ All areas within the Stage II excavation limits were excavated to depths as directed, which removed all highly organic material, loose rock and any other objectionable foundation materials as determined by the Contracting Officer.

Approximate ground surface

Approximate Limits of Excavation

Bethany Falls Limestone

Hushpuckney Shale

Snider Limestone

Mound City Shale

Critzler Limestone

Ladore Shale

Middle Creek Limestone

Pleasanton Group

Approximate top of bedrock

0+00 1+00 2+00

**SECTION B**  
29/29

Symbol

Design

Drawn by

Checked by

Submitted by

Scale: AS SHOWN

Date

Draw No

File No

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Little Blue River, Missouri  
Longview Lake  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
RIGHT ABUTMENT SECTION  
AND PLAN OF EXCAVATION

Designed by  
R G D

Drawn by  
J W P

Checked by  
R G D / W B E

Submitted by  
R G D

Scale: AS SHOWN

Date

Draw No

File No  
RBL-3-1429

Cut in slope in overburden

Slope in Wea and Block Members

Temporary protective soil cover removed

Cut slope in winterset Limestone

Cut slope in Bethany Falls Limestone

APPROXIMATE EXCAVATION LIMITS

APPROXIMATE STAGE II ABUTMENT EXCAVATION LIMITS

Excavated IV on Q5H through Bethany Falls Limestone

Not steeper than IV on Q5H

Constructed bench as necessary to excavate Bethany Falls Limestone

CORPS OF ENGINEERS SURVEY - JUNE, 1984

PLATE NO 29

- Notes:**

  - ① Limits of Stage II excavation shown are approximate. Actual vertical and horizontal limits depend upon conditions encountered and were as directed by the Contracting Officer.
  - ② Stage I rock surfaces received foundation cleanup or preparation as directed by the Contracting Officer and removal of temporary protective soil cover.
  - ③ All areas within the Stage II excavation limits were excavated to depths as directed, which removed all highly organic material, loose rock and any other objectionable foundation materials as determined by the Contracting Officer.

**SECTION B**

Approximate ground surface  
Approximate Limits of Excavation  
Bethany Falls Limestone  
Hushpuckney Shale  
Snider Limestone  
Mound City Shale  
Critzler Limestone  
Ladore Shale  
Middle Creek Limestone  
Pleasanton Group  
Approximate top of bedrock

Approximate embankment toe trace

Cut in slope in overburden

Slope in Wea and Block Members

Temporary protective soil cover removed

Cut slope in winterset Limestone

Cut slope in Bethany Falls Limestone

APPROXIMATE STAGE II ABUTMENT EXCAVATION LIMITS

Excavated IV on 0.5H through Bethany Falls Limestone

Not steeper than IV on 0.5H

Constructed bench as necessary to excavate Bethany Falls Limestone

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

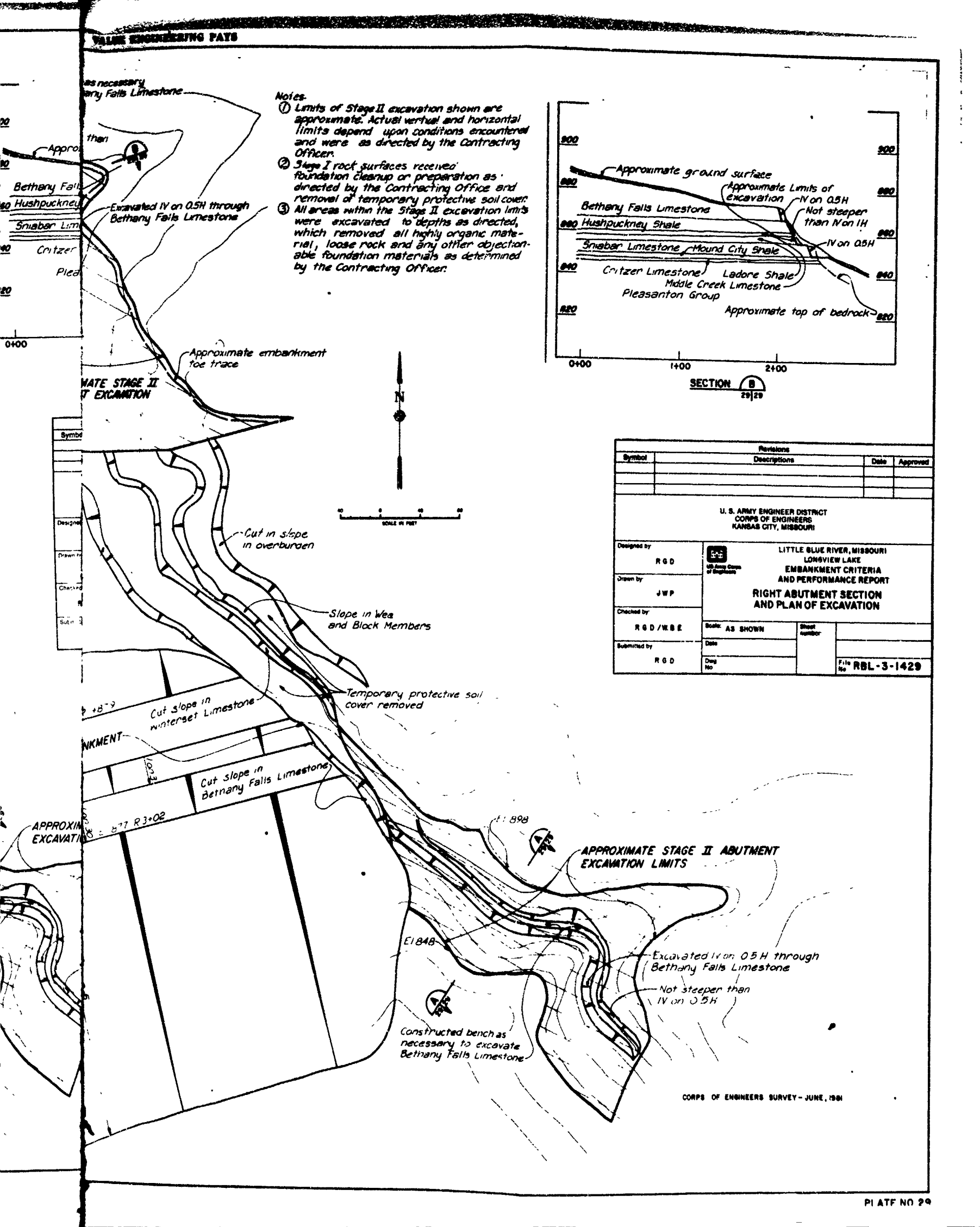
Little Blue River, Missouri  
Longview Lake  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
RIGHT ABUTMENT SECTION  
AND PLAN OF EXCAVATION

Designed by: RGD  
Drawn by: JWP  
Checked by: RGD/WBE  
Submitted by: RGD

Scale: AS SHOWN  
Date:   
Draw No:   
Sheet number:   
File No: RBL-3-1429

CORPS OF ENGINEERS SURVEY - JUNE, 1984

PLATE NO 29



**VALUE ENGINEERING PAYS**

**Notes:**

- Limits of Stage II excavation shown are approximate. Actual vertical and horizontal limits depend upon conditions encountered and were as directed by the Contracting Officer.
- Stage I rock surfaces received foundation cleanup or preparation as directed by the Contracting Office and removal of temporary protective soil cover.
- All areas within the Stage II excavation limits were excavated to depths as directed, which removed all highly organic material, loose rock and any other objectionable foundation materials as determined by the Contracting Officer.

**SECTION B**

Revisions			
Symbol	Description	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: RGD  
Drawn by: JWP  
Checked by: RGD/WBE  
Submitted by: RGD

**LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
RIGHT ABUTMENT SECTION  
AND PLAN OF EXCAVATION**

Scale: AS SHOWN  
Date:      Sheet number:  
Drawing No.:      File No: RBL-3-1429

CORPS OF ENGINEERS SURVEY - JUNE, 1984

PLATE NO 29

**VALUE ENGINEERING PAYS**

**Notes:**

- Limits of Stage II excavation shown are approximate. Actual vertical and horizontal limits depend upon conditions encountered and were as directed by the Contracting Officer.
- Stage I rock surfaces received foundation cleanup or preparation as directed by the Contracting Office and removal of temporary protective soil cover.
- All areas within the Stage II excavation limits were excavated to depths as directed, which removed all highly organic material, loose rock and any other objectionable foundation materials as determined by the Contracting Officer.

**Approximate ground surface**  
**Approximate Limits of Excavation**  
 Bethany Falls Limestone  
 Hushpuckney Shale  
 Snider Limestone  
 Mound City Shale  
 Critzer Limestone  
 Ladore Shale  
 Middle Creek Limestone  
 Pleasanton Group  
 Approximate top of bedrock

**SECTION B**  
 29/29

Revisions			
Symbol	Description	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
 CORPS OF ENGINEERS  
 KANSAS CITY, MISSOURI

Designed by: RGD  
 Drawn by: JWP  
 Checked by: RGD/WBE  
 Submitted by: RGD

**LITTLE BLUE RIVER, MISSOURI  
 LONGVIEW LAKE  
 EMBANKMENT CRITERIA  
 AND PERFORMANCE REPORT  
 RIGHT ABUTMENT SECTION  
 AND PLAN OF EXCAVATION**

Scale: AS SHOWN  
 Date:      Sheet number:  
 Drawing No.:      File No: RBL-3-1429

**APPROXIMATE STAGE II ABUTMENT EXCAVATION LIMITS**

Excavated IV on QSH through Bethany Falls Limestone  
 Not steeper than IV on QSH

Constructed benches necessary to excavate Bethany Falls Limestone

Cut slope in winterset Limestone  
 Cut slope in Bethany Falls Limestone

Slope in Wea and Block Members

Temporary protective soil cover removed

Cut in slope in overburden

Approximate embankment toe trace

Excavated IV on QSH through Bethany Falls Limestone

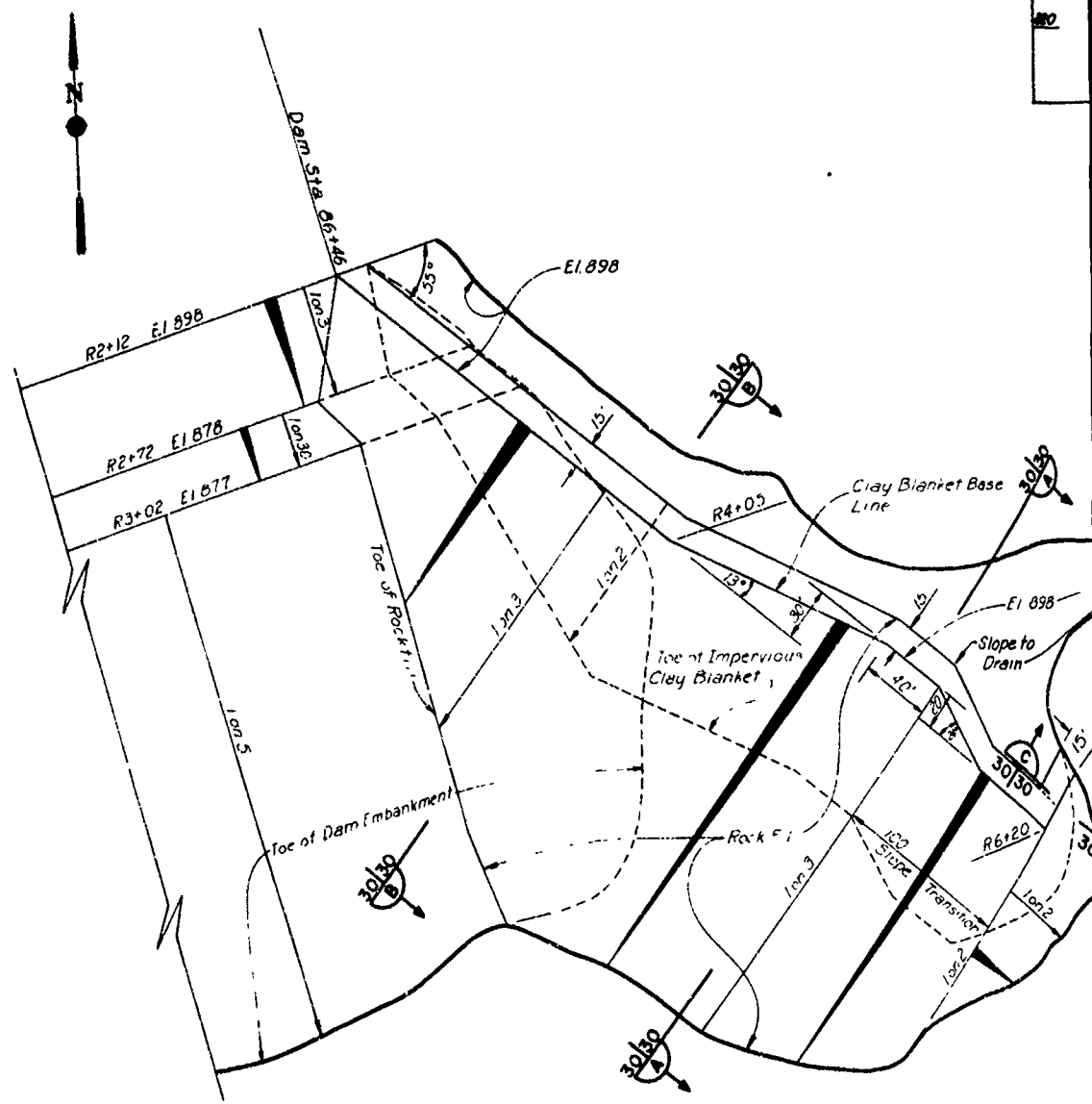
Bethany Falls Limestone  
 Hushpuckney Shale  
 Snider Limestone  
 Mound City Shale  
 Critzer Limestone  
 Ladore Shale  
 Middle Creek Limestone  
 Pleasanton Group

Approximate top of bedrock

SCALE IN FEET

CORPS OF ENGINEERS SURVEY - JUNE, 1984

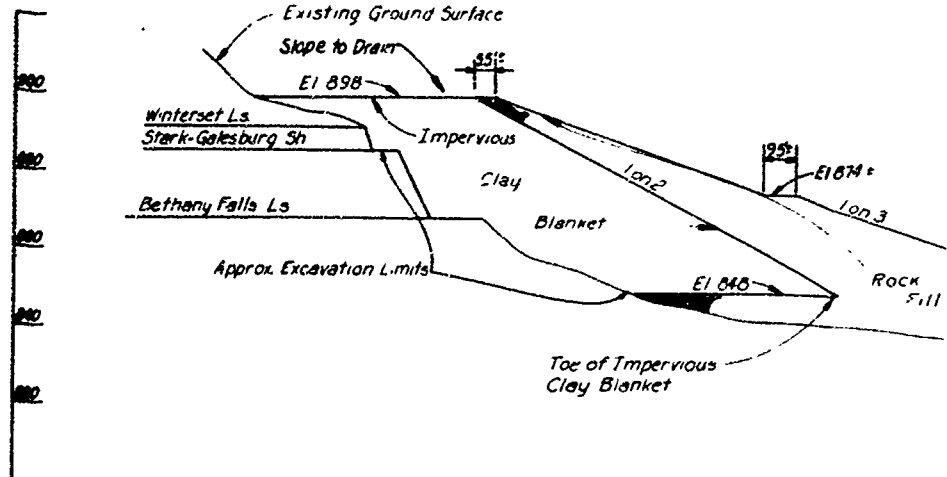
PLATE NO 29



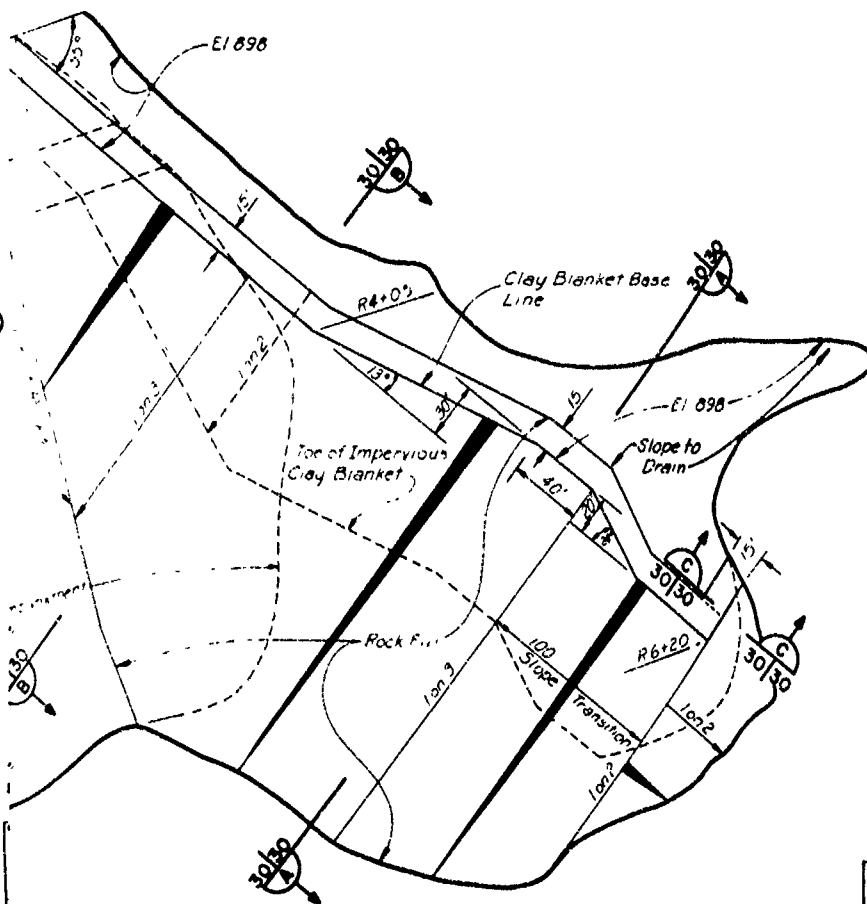
PLAN OF UPSTREAM CLAY BLANKET



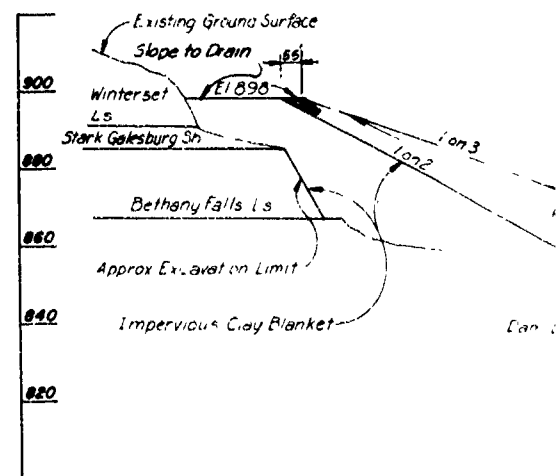
Wint  
Stark  
Beth



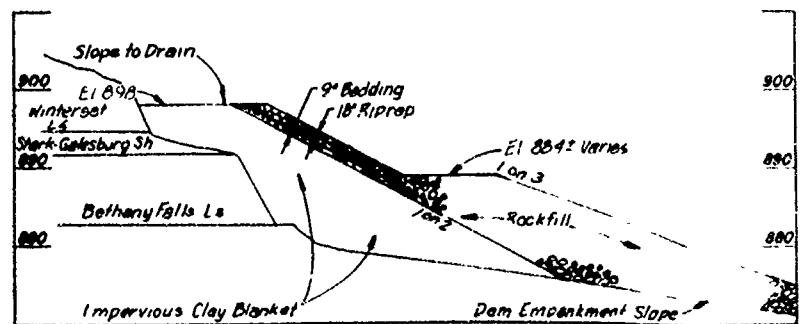
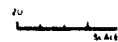
UPSTREAM CLAY BLANKET SECTION A



PLAN OF UPSTREAM CLAY BLANKET



UPSTREAM CLAY BLANKET SECTION B



UPSTREAM CLAY BLANKET SECTION C

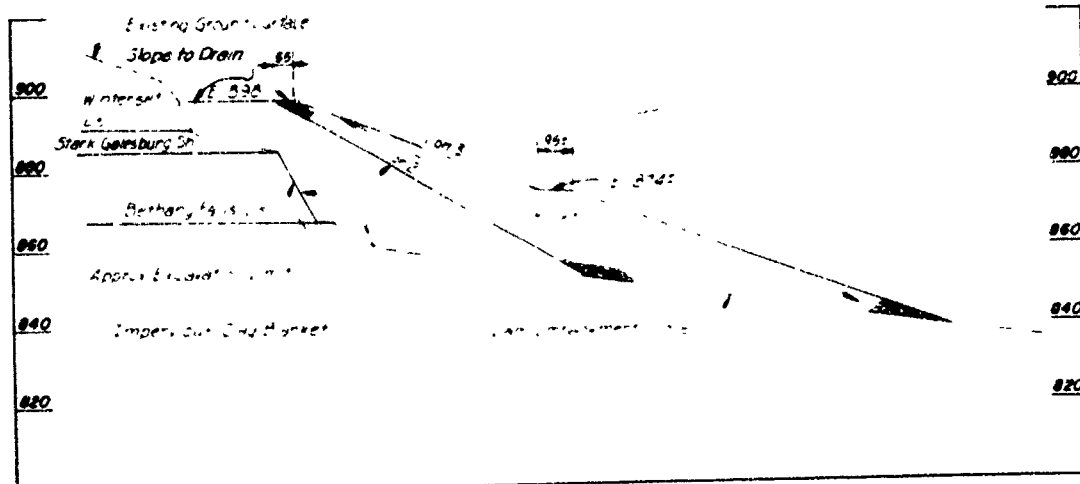


30130

UPSTREAM CLAY BLANKET SECTION

30130

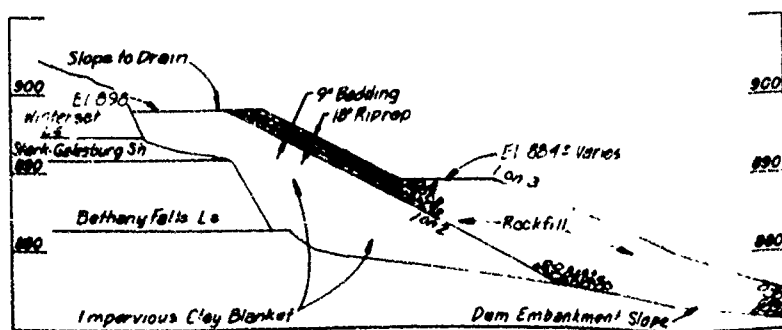
SCALE IN FEET



UPSTREAM CLAY BLANKET SECTION

30130

SCALE IN FEET



UPSTREAM CLAY BLANKET SECTION

30130

SCALE IN FEET

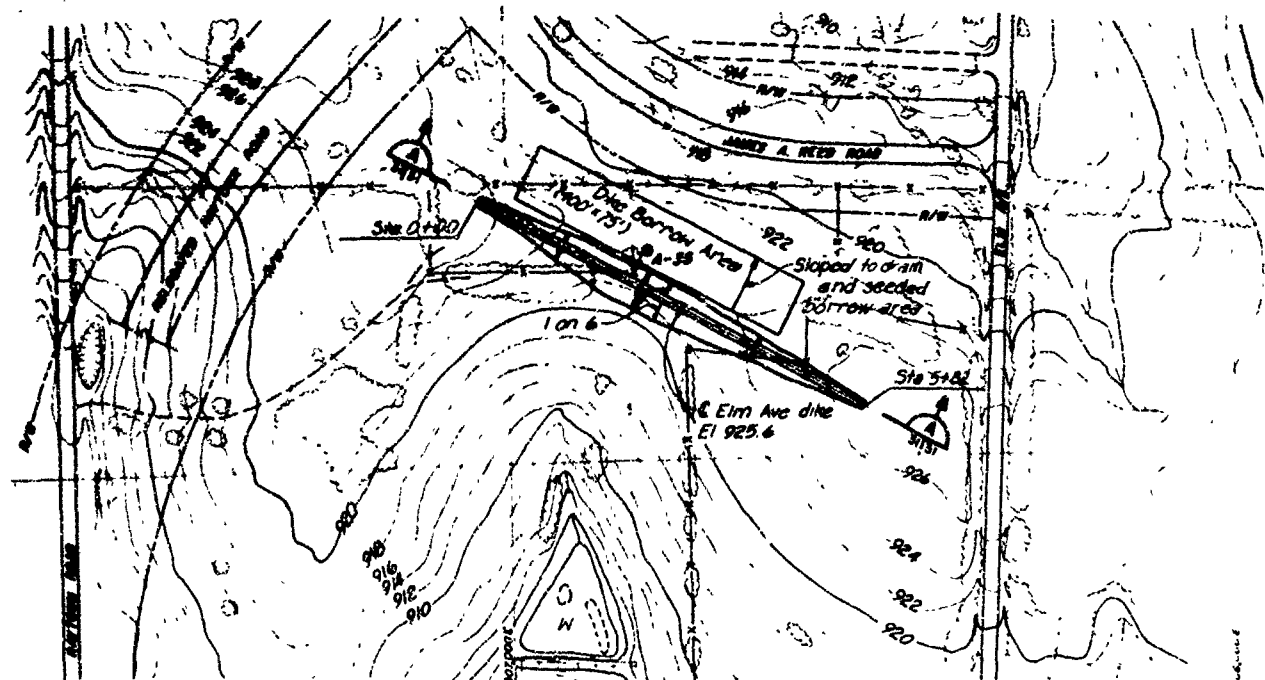
Revisions			
Symbol	Descriptions	Date	Approved

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: R.G.D.  
Drawn by: J.W.T.  
Checked by: R.G.D.  
Approved by: R.G.D.

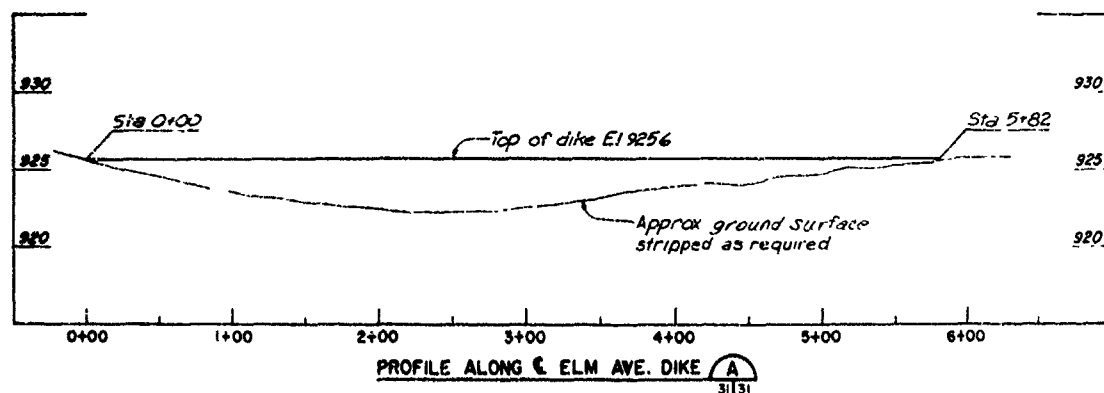
LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
RIGHT ABUTMENT  
UPSTREAM IMPERVIOUS  
CLAY BLANKET PLAN AND SECTIONS

Sheet Number: 2046  
Date: 2/5/64  
RBL-3-1430

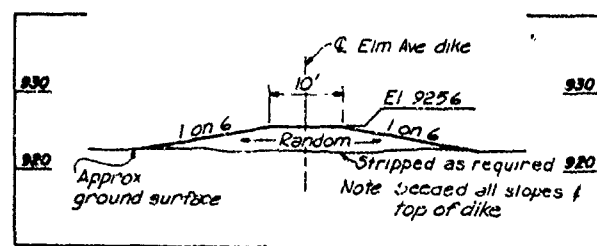


ELM AVE. DIKE PLAN  
SCALE IN FEET

ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929



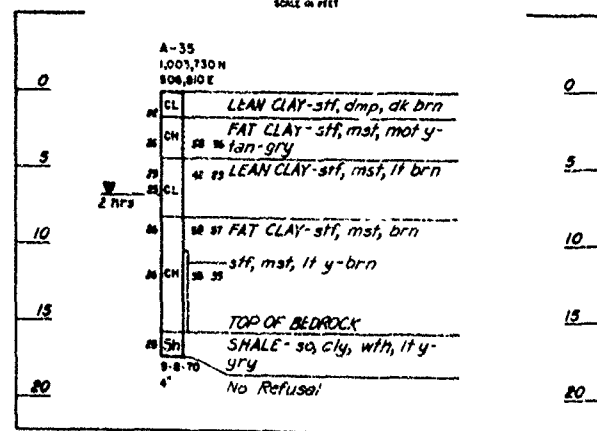
PROFILE ALONG C. ELM AVE. DIKE A  
31/31



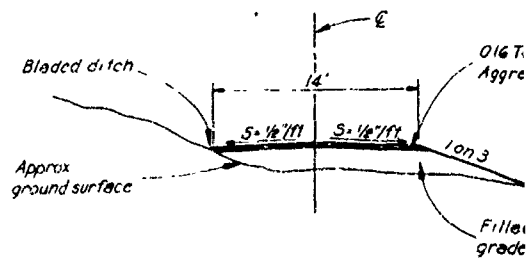
ELM AVE. DIKE-TYPICAL SECTION  
SCALE IN FEET

ELM AVE. DIKE ALINEMENT DATA				
POT. STATION	LATITUDE	DEPARTURE	AZIMUTH	DIST
0+00 BEGIN DIKE	1,003,643.793	507,035.036	118° 02' 20"	582.00
5+82 END DIKE	1,003,937.374	506,521.346		

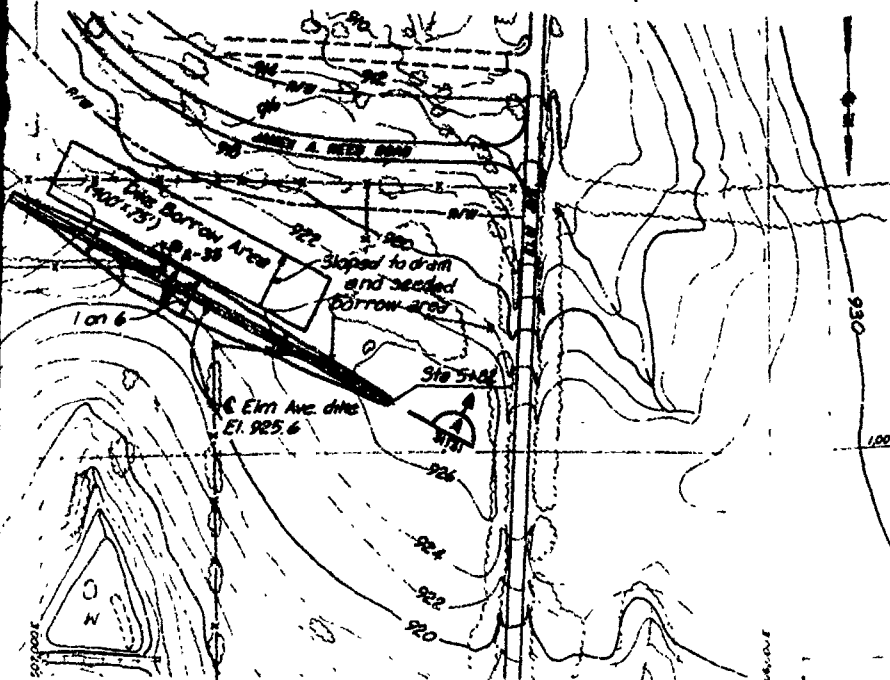
DEPTH IN FEET



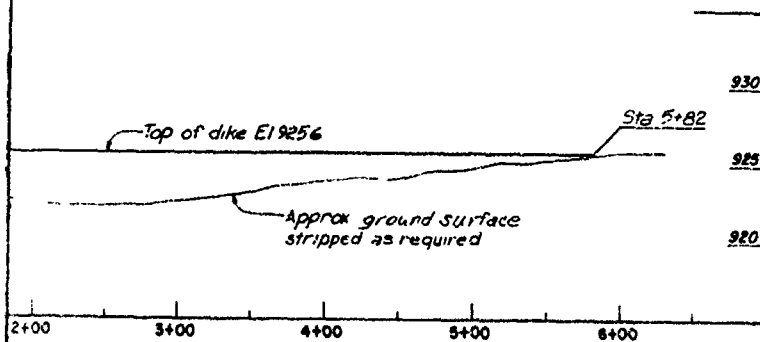
ELM DIKE BORROW AREA-LOG OF BORING



EMERGENCY ACCESS LANE  
Typical Fill Section  
Scale 1"=5'



ELM AVE. DIKE PLAN

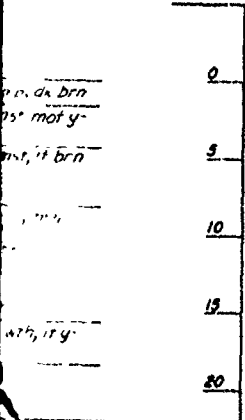


PROFILE ALONG ELM AVE. DIKE

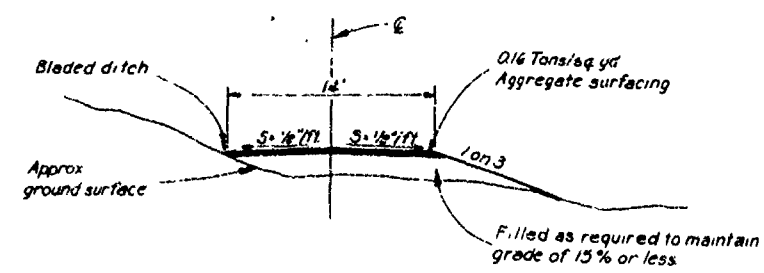
Elm Ave dike  
El. 925.6  
1 on 6  
Stripped as required  
Sloped all slopes to  
top of dike

ELM AVE. DIKE ALINEMENT DATA				
POT. STATION	LATITUDE	DEPARTURE	AZIMUTH	DIST
0+00 BEGIN DKE	1,003,663.793	807,035.036		
5+82 END DKE	1,003,937.374	806,521.346	118° 02' 20"	882.00

L SECTION



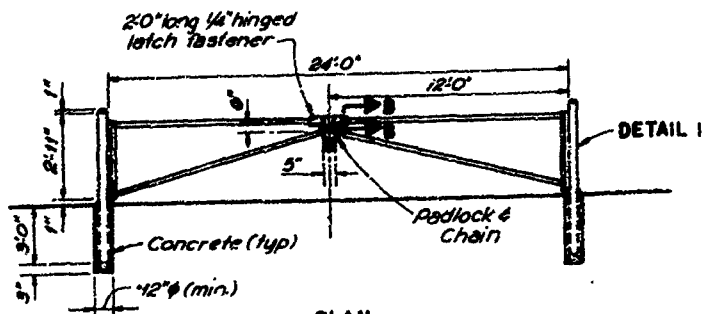
LOG OF BORING



EMERGENCY ACCESS LANE

Typical Fill Section

Scale 1"=5'

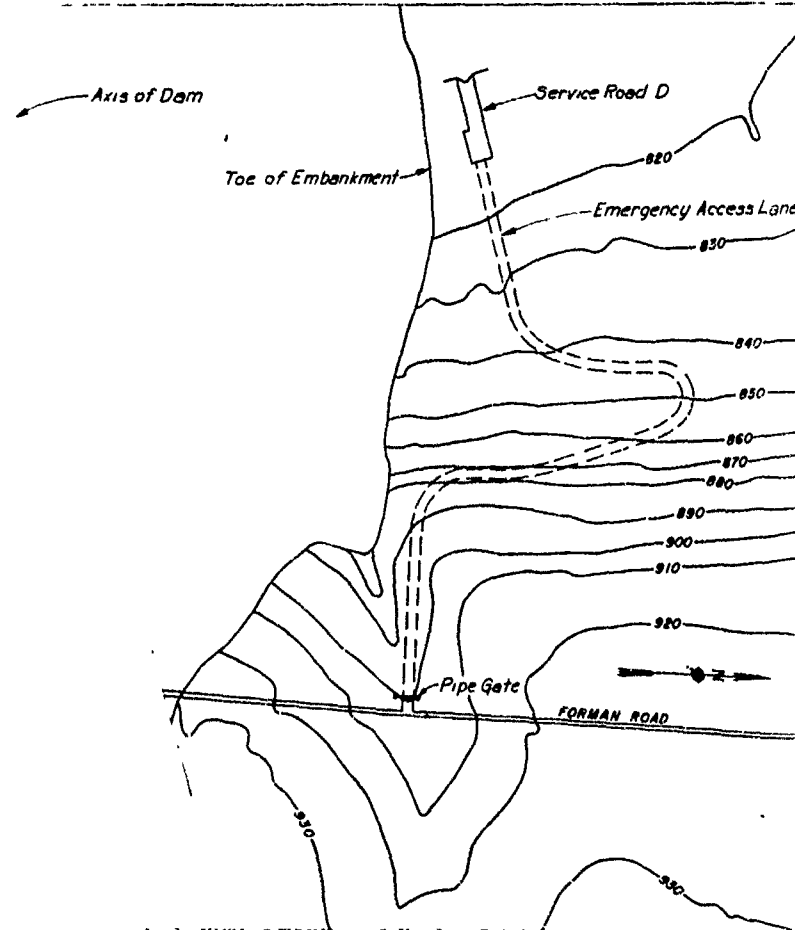


PLAN  
PIPE GATE

PIPE GATE DETAILS

Not to Scale

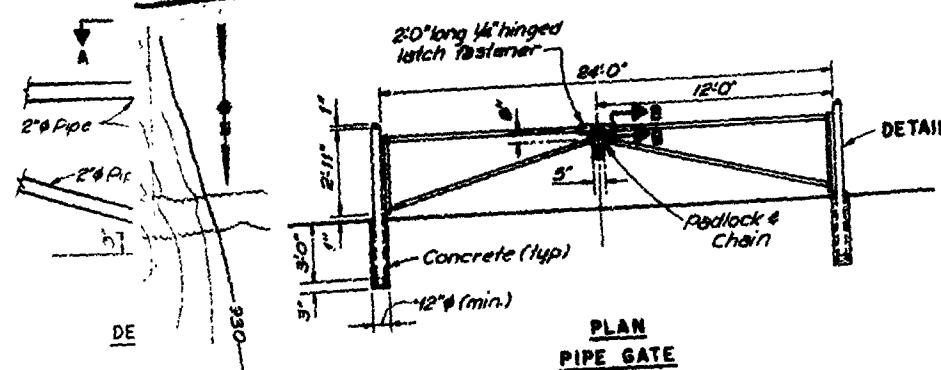
NOT:  
1. A,  
2. F  
at  
3. P  
pr  
e)



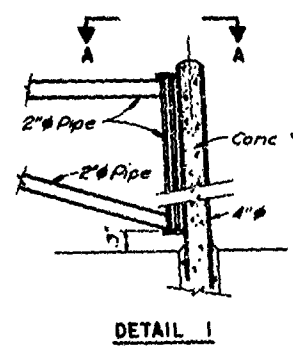
EMERGENCY ACCESS LANE PLAN

Scale 1"=100'

# EMERGENCY PAYS



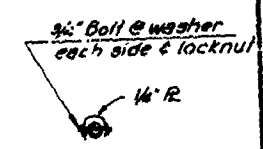
PLAN  
PIPE GATE



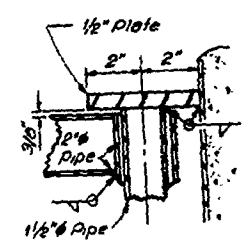
DETAIL I



SECTION A



SECTION B

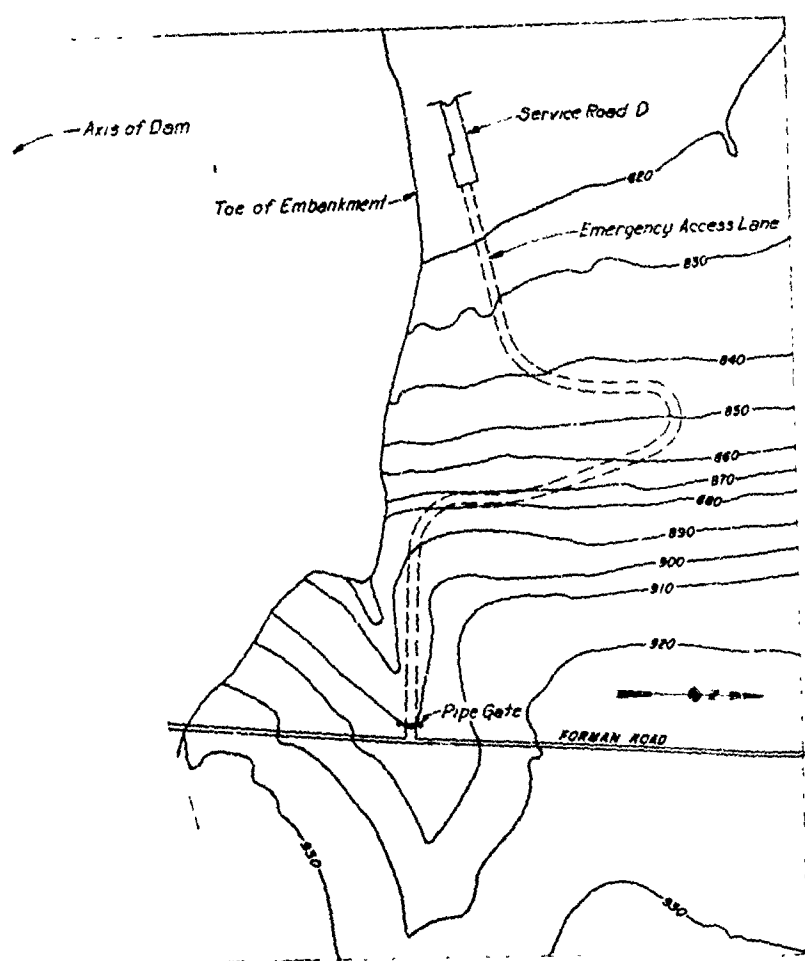


SECTION C  
(Typical @ Top & Bottom)

- NOTES:
1. All pipe Sch. 40
  2. Pipe Gate completely welded at all joints and watertight.
  3. Painted with one coat red lead primer and two coats yellow exterior enamel.

- NOTES:
1. All pipe Schedule 40 Steel
  2. Pipe Gate completely welded at all joints and watertight.
  3. Painted with one coat red lead primer and two coats yellow exterior enamel.

PIPE GATE DETAILS  
Not to Scale

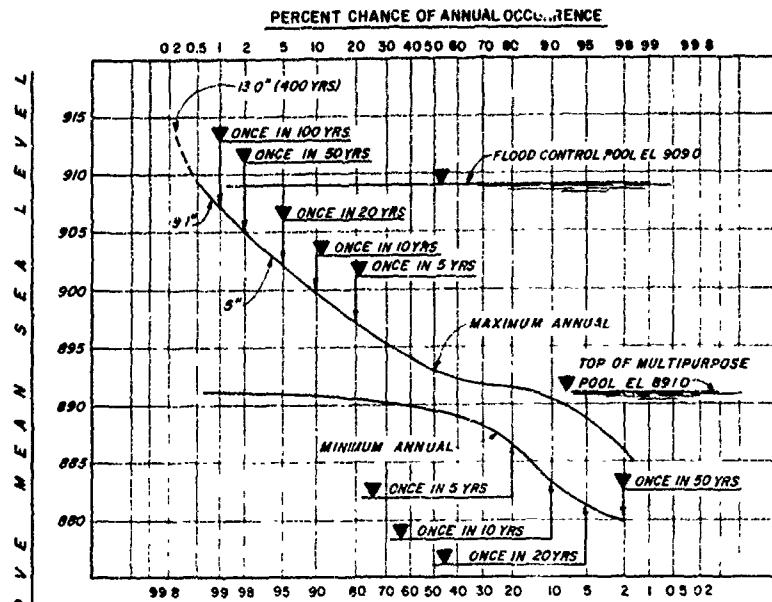


EMERGENCY ACCESS LANE PLAN  
Scale 1" = 100'

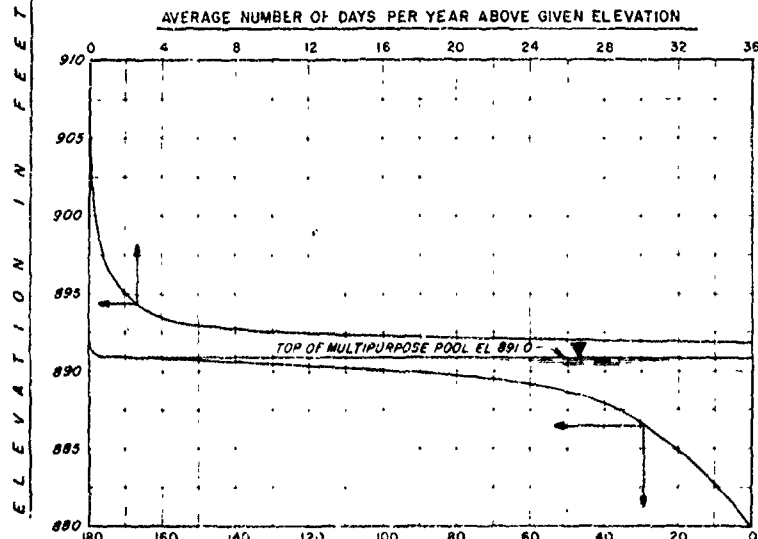
2 1/2 Tons/sq. yd.  
Aggregate surfacing

as required to maintain  
grade of 15% or less

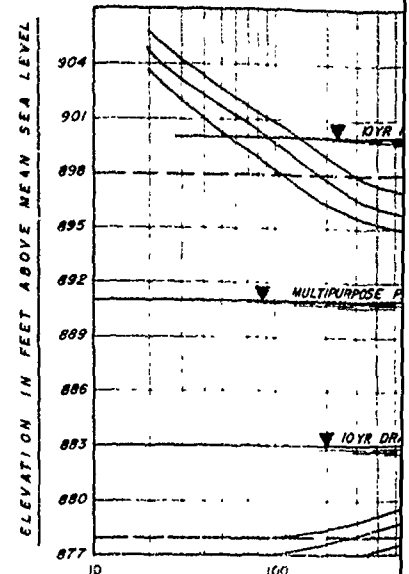
Revisions		Date	Approved
Symbol	Descriptions		
<p>U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p>			
Designed by	R G D	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</p>	
Drawn by	J W P	<p>ELM AVENUE DIKE ACCESS LANE AND PIPE GATE DETAILS</p>	
Checked by	R.S.D./W.B.E.	Scale	AS SHOWN
Submitted by	R G D.	Date	
		Dwg No.	
		Sheet Number	
		File No.	RBL-3-1431



POOL ELEVATION - EXCEEDANCE FREQUENCY CURVE



AVERAGE NUMBER OF DAYS PER YEAR BELOW GIVEN ELEVATION  
POOL ELEVATION - DURATION CURVE



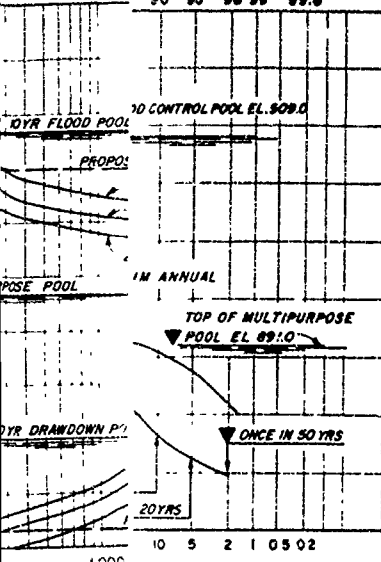
DAYS PER 100 YRS THAT POOL IS FULL

WIND VELOCITY - FREQ			
DIRECTION	FREQUENCY YRS	TOPEKA, KS MPH	CS
SSW	20	20	
S	20	20	
SSW	20	20	
SW	20	20	
SSW	20	20	
S	20	20	
SSW	20	20	
SW	20	20	
SSW	20	20	
S	20	20	
SSW	20	20	
SW	20	20	

Notes:  
Average wind speed  
data from Post 1st  
at a point near the  
pool.

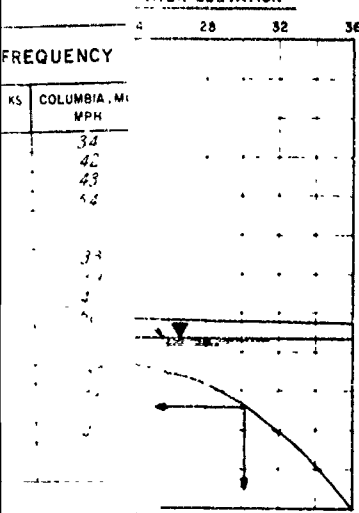
# CURRENCE

90 95 98 99 99.8

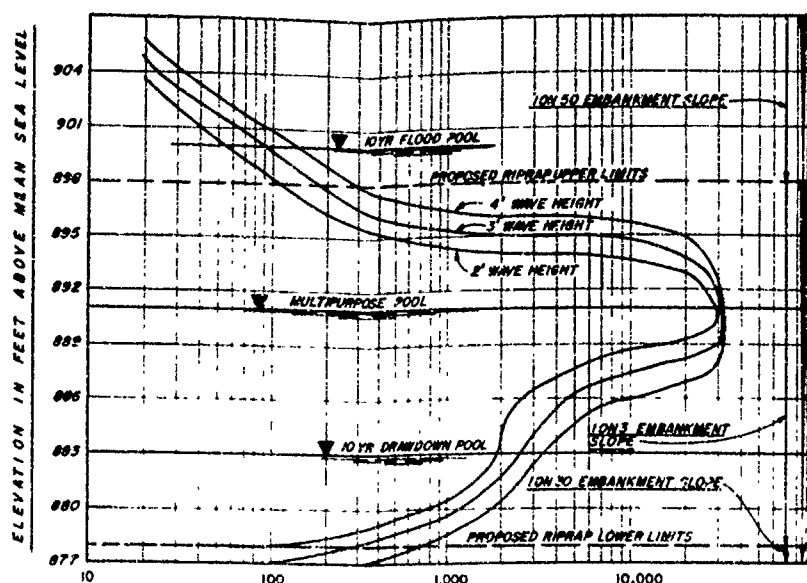


POOL IS WITHIN  
FREQUENCY CURVE

GIVEN ELEVATION



GIVEN ELEVATION  
CURVE

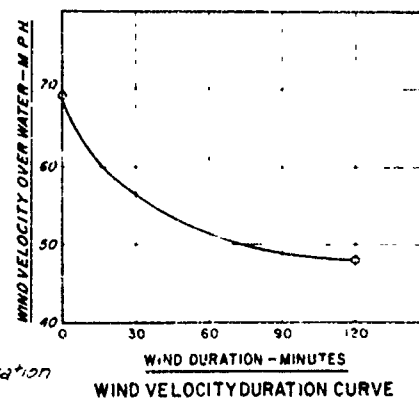


DAYS PER 100 YRS THAT POOL IS WITHIN ONE WAVE HEIGHT OF EACH ELEV

## WIND VELOCITY-FREQUENCY CHART

DIRECTION	FREQUENCY YRS	TOPEKA, KS MPH	COLUMBIA, MO MPH	KANSAS CITY, MO MPH
SSE	100	49	34	31
S	100	56	42	38
SSW	100	45	43	41
SW	100	51	44	40
SSW	5	42	33	29
S	5	47	37	36
SSW	5	42	33	29
SW	5	47	37	36
SSE	5	42	33	29
S	5	47	37	36
SSW	5	42	33	29
SW	5	47	37	36

Notes  
Average wind speed  
Data from first 10 years of bureau recording station  
at airport near residence



Notes  
Wind velocity computed from  
data from first 10 years of bureau recording station  
at airport near residence

## RIPRAP ST

LOCATION	FETCH	DESIGN WIND	DESIGN
STA	ELEV	SLOPE	LENGTH
89+30	920	1 on 3	14 SW
95+30	920	1 on 3	12 SW
101+90	920	on 3	8 SW
89+90	891	on 3	66 SW
95+30	891	1 on 3	67 SW
101+90	891	on 3	40 SW
99+20	891	on 3	67 SW

Note  
1 Fetch length in miles  
2 A rough estimate of the wind velocity was obtained from  
upper 10 of dam...  
3. From Design...  
4. From Design...

## CRITICAL WIND VELOCITY

WIND VELOCITY MPH	WIND
48	
49	
50	
51	
52	
53	
54	
55	
56	
57	
58	
59	
60	
61	
62	
63	
64	
65	
66	
67	
68	
69	
70	
71	
72	
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92	
93	
94	
95	
96	
97	
98	
99	
100	

Symbol

Designed by

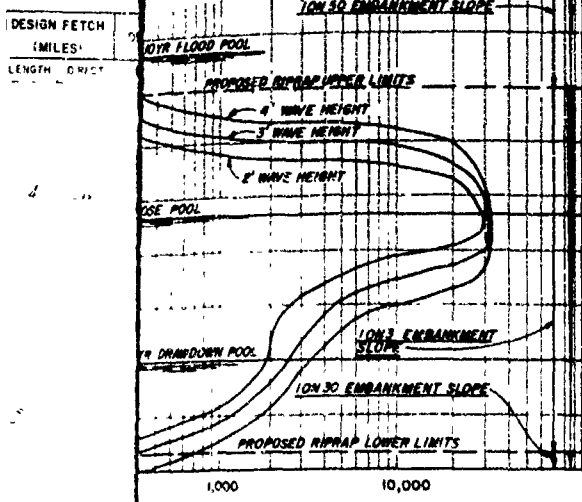
Drawn by

Check by

Scale

Date

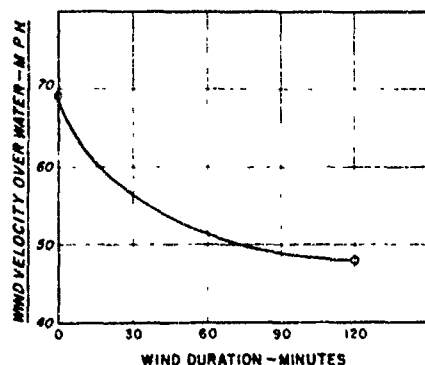
AP STUDY



POOL IS WITHIN ONE WAVE HEIGHT OF EACH ELEV

DVELOCITY

FREQUENCY CHART		
RIPRAP AT ELEV	COLUMBIA, MO MPH	KANSAS CITY, MO MPH
WIND DURA MIN	34	31
	40	38
	43	41
	46	44
	54	50
	56	53
	59	56
	63	60



WIND VELOCITY DURATION CURVE

Note  
Wind velocity computed from E.T.C. 10-2-12  
(29 Nov. 1963) converted to over water velocities  
by ratio of 1.0113

RIPRAP STUDY

(SPECIFIC GRAVITY = 2.6)

SPECIFIC GRAVITY = 2.65

LOCATION		FETCH		DESIGN WIND		DESIGN FETCH (MILES)		DESIGN WAVE		RIPRAP				
STA	ELEV	SLOPE	LENGTH	DIRECT	DURATION (MIN)	SPEED (MPH)	LENGTH	DIRECT	H <sub>s</sub>	H <sub>D</sub>	W <sub>o</sub>	W <sub>o</sub> PROPOSED	THICK	S F
89130	920	1 on 3	74	SW										
95130	920	1 on 3	12	SW	30	56	14	SW	29	41	117	100	18"	See note
101190	920	1 on 3	8	SW										
89190	891	1 on 3	66	SW										
95130	891	1 on 3	67	SW										
101190	891	1 on 3	46	S	20	59	07	SW	22	31	51	100	18"	13
98120	891	1 on 3	67	SW										

Note  
1 Fetch length in miles.  
2 A reduced thickness of riprap was utilized for the upper 10' of dam in accordance with "Criteria for Riprap Design" Missouri River Division June 1964

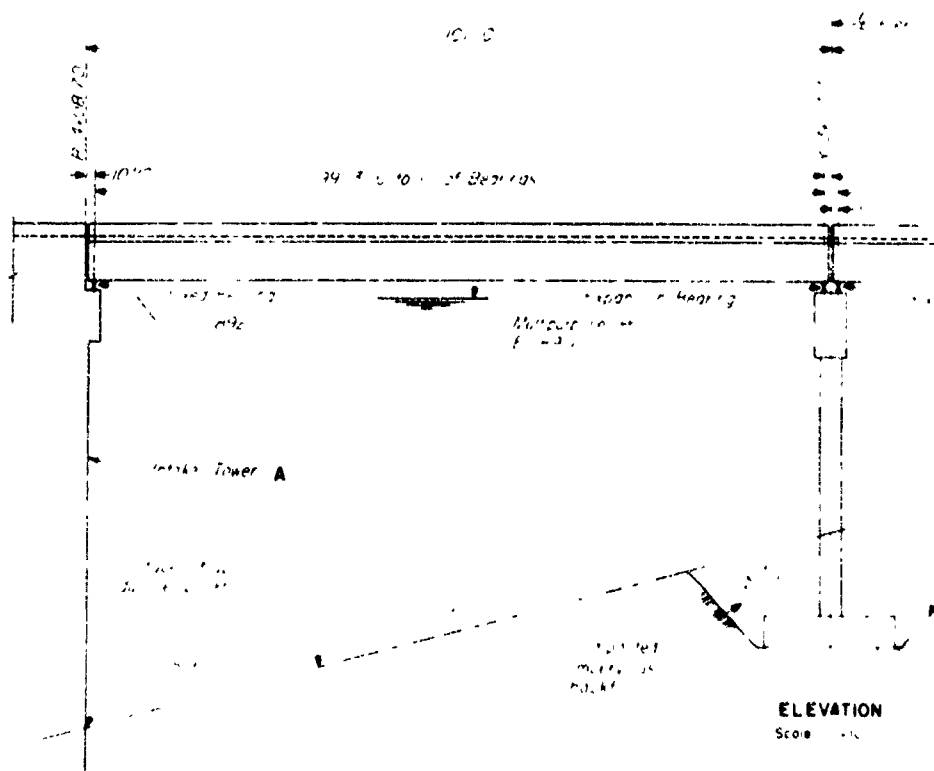
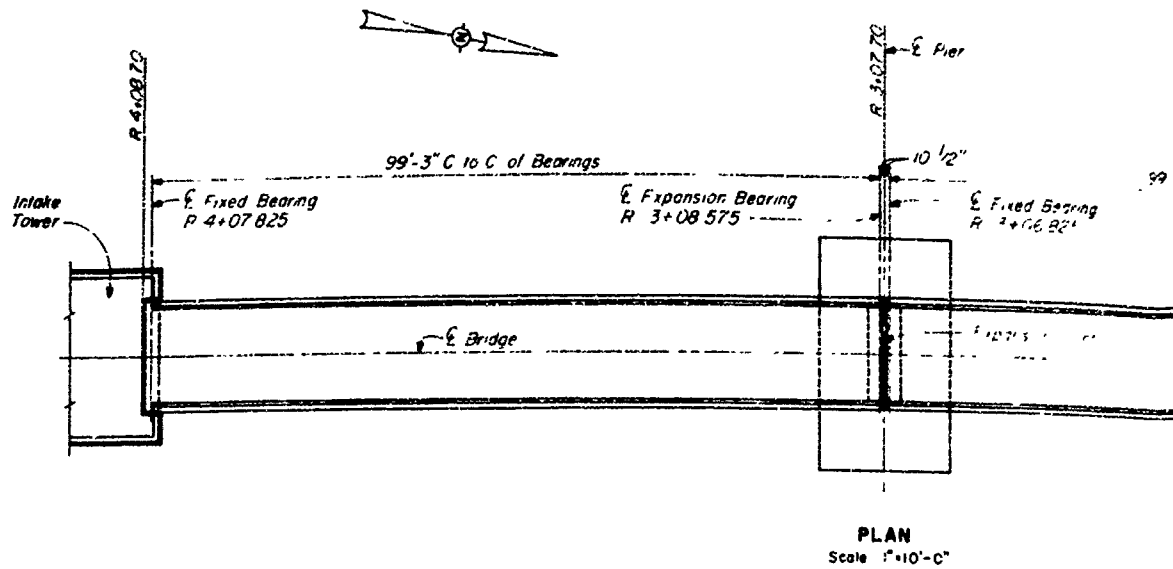
CRITICAL WIND VELOCITY - DURATION CHART

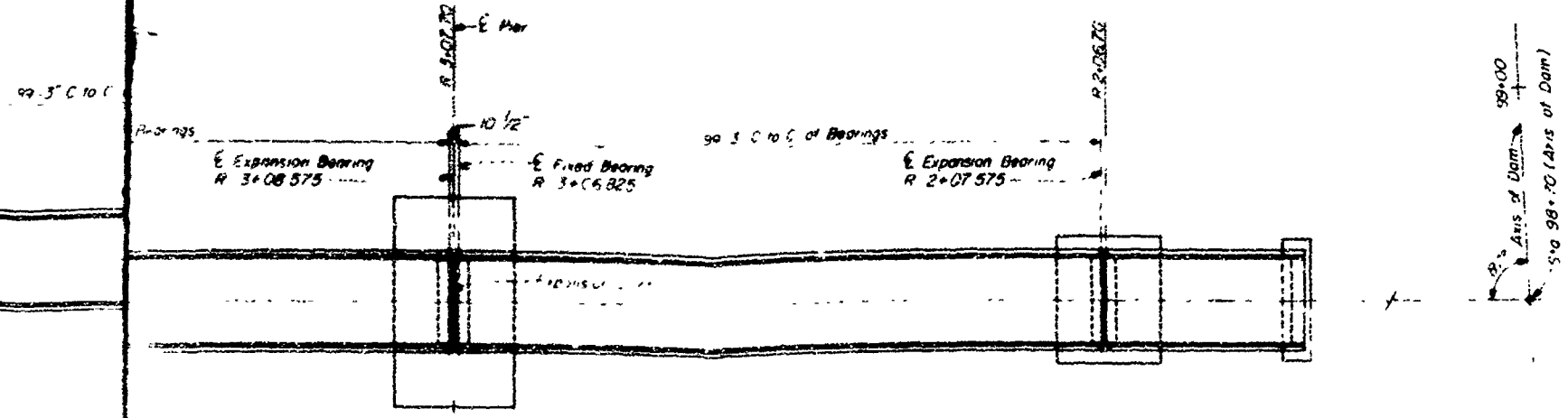
RIPRAP AT ELEV 891		
WIND VELOCITY M.P.H.	WIND DURATION MIN	FORECAST HS - FT
40	120	5 (Fetch limited)
49	90	18 (Fetch limited)
51	60	18 (Fetch limited)
53	50	20 (Fetch limited)
54	40	20 (Fetch limited)
56	30	21 (Fetch limited)
59	20	22 (Fetch limited)
63	10	23 (Duration limit)
RIPRAP AT ELEV 920		
40	120	20 (Fetch limited)
49	90	26 (Fetch limited)
51	60	16 (Fetch limited)
53	50	18 (Fetch limited)
54	40	18 (Fetch limited)
56	30	29 (Fetch limited)
59	20	24 (Duration limit)
63	10	22 (Duration limit)

XX indicates critical wind velocity and duration

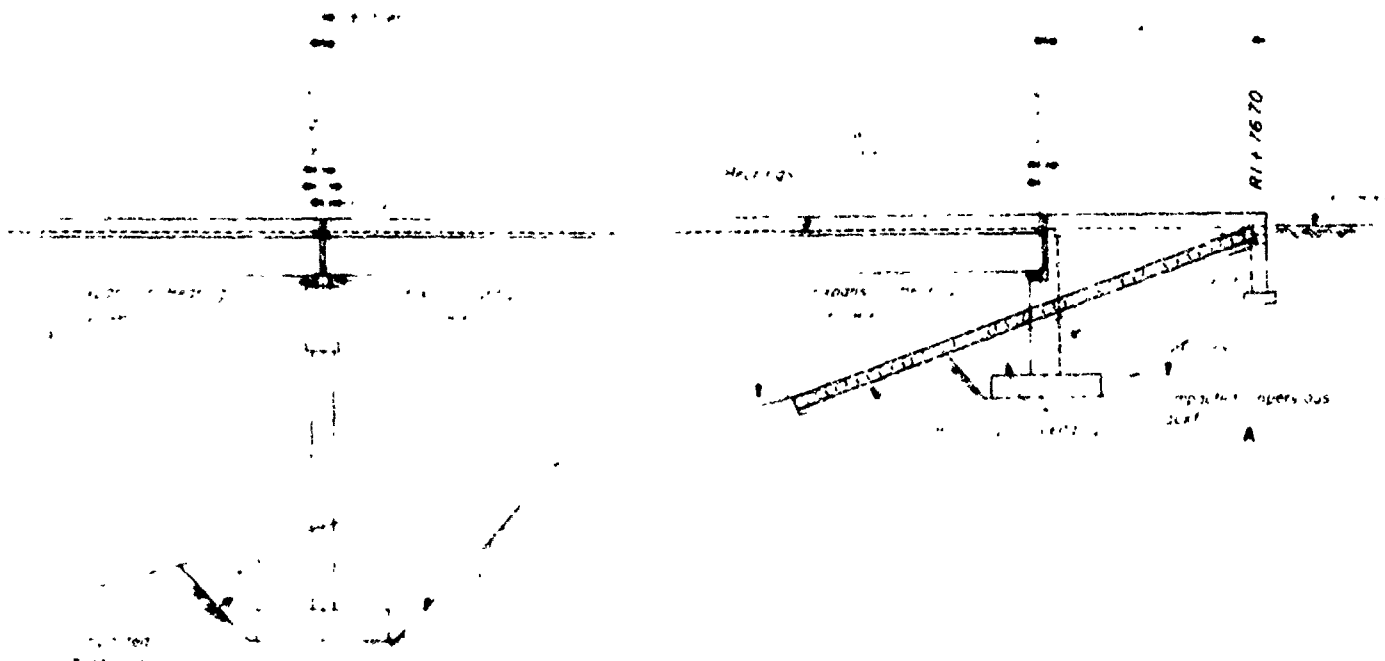
Revisions			
Symbol	Descriptions	Date	Approved
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by R.G.D.	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT  SLOPE PROTECTION STUDY		
Drawn by J.E.B.			
Checked by R.G.D./W.B.E.			
Submitted by R.G.D.	Scale AS SHOWN	Sheet number	File No. RBL-3-1432







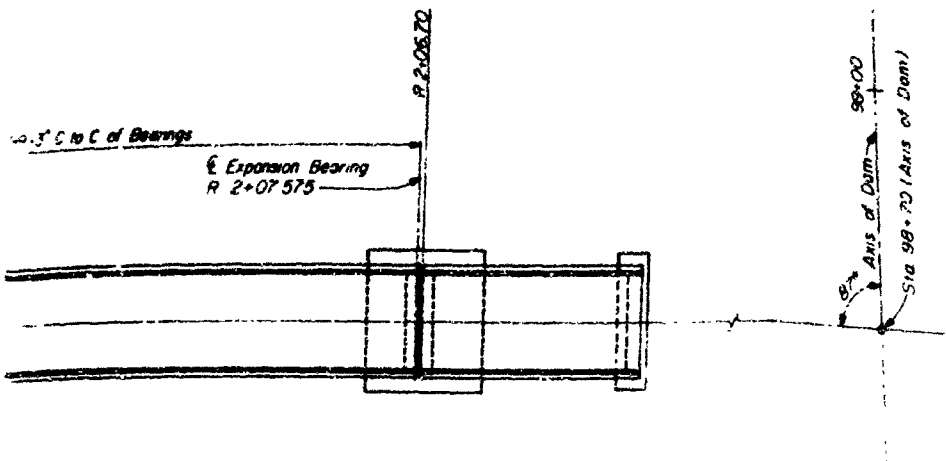
PLAN  
Scale 1"=10'-0"



ELEVATION  
Scale 1"=10'-0"

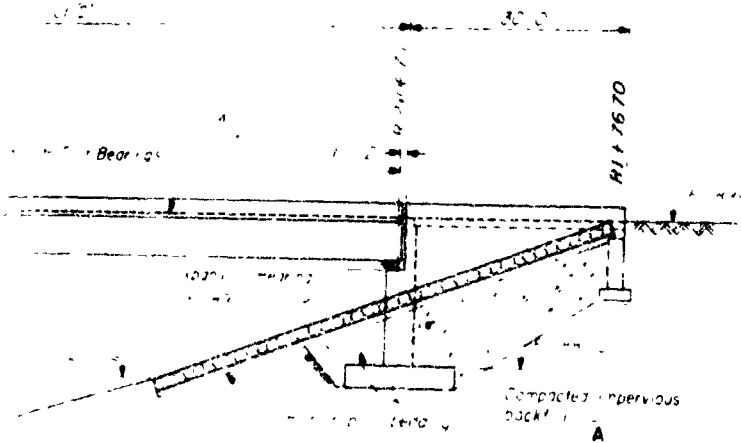
GENE  
1.  
2. 1/4  
3.  
4.  
5.  
6. 8'  
7.

Sym  
+  
1/4  
1/2  
3/4  
1



# GENERAL NOTES

1. Design Specification AASHTO 1977 Edition
2. Live loads H 5-44 or normal stresses
3. Multiple spans carrying 35 tons carrying 90 kips, live load of 90 kips without impact
4. All exterior corners chamfered 3/4"
5. All reinforcement bars lapped
6. Bridge stations are measured along centerline of Outlet Works with starting point at Intake Tower



## GENERAL NOTES - BRIDGE

1. Design Specification AASHTO 1977 Edition
2. Live loads H 5-44 or normal stresses
3. Multiple spans carrying 35 tons carrying 90 kips, live load of 90 kips without impact
4. All exterior corners chamfered 3/4"
5. All reinforcement bars lapped
6. Bridge stations are measured along centerline of Outlet Works with starting point at Intake Tower

Symbol	Revisions	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: RGD

Drawn by: PRL

Checked by: RGD/WBE

Submitted by: RGD

**LITTLE BLUE RIVER, MISSOURI**  
**LONGVIEW LAKE**  
**EMBANKMENT CRITERIA**  
**AND PERFORMANCE REPORT**

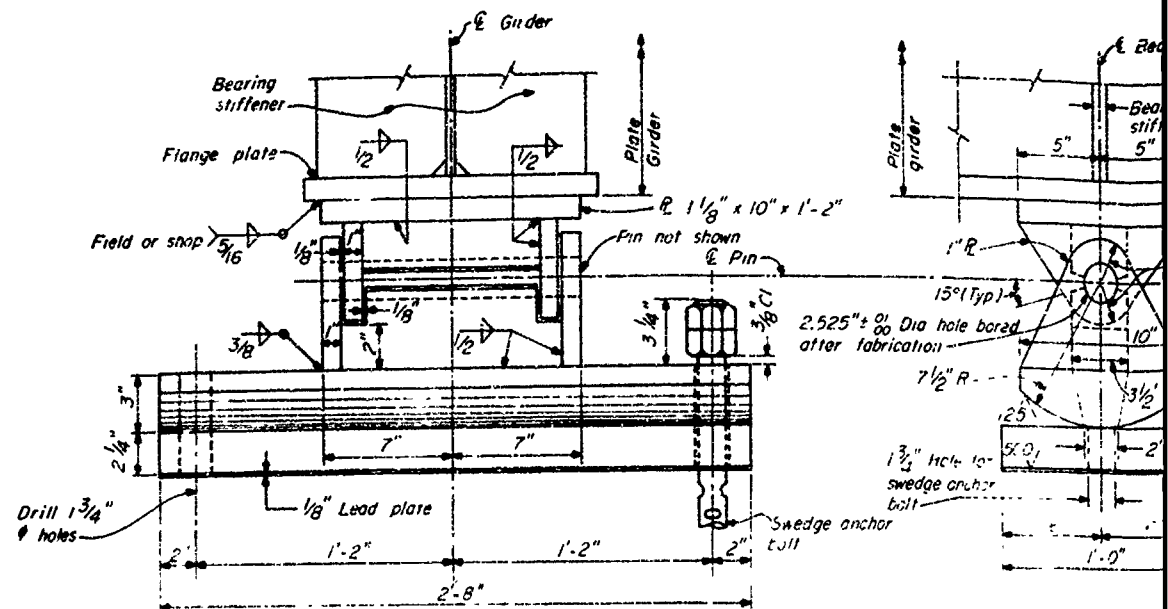
**SERVICE BRIDGE**  
**PLAN AND ELEVATION**

Scale: AS SHOWN

Sheet number: \_\_\_\_\_

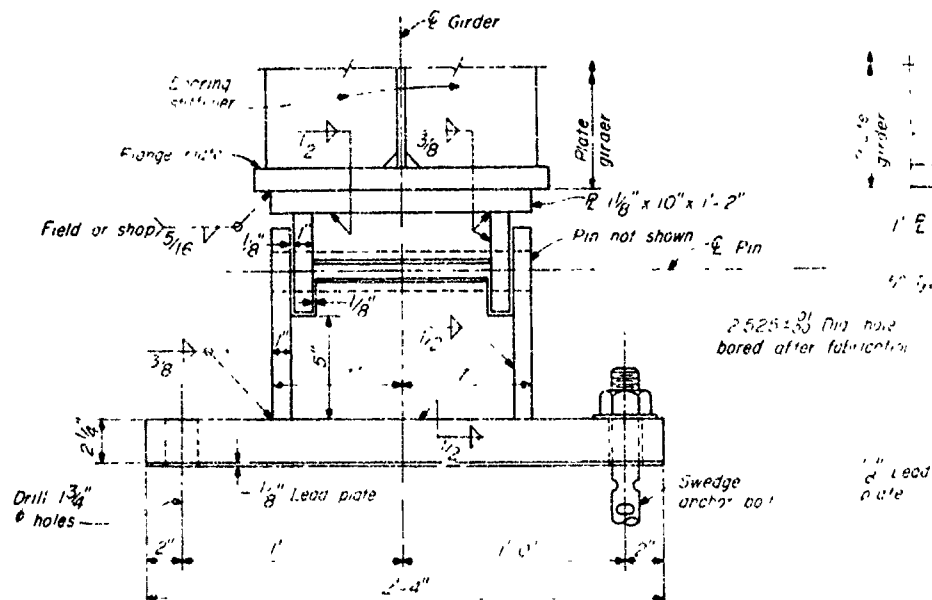
Date: \_\_\_\_\_

File No: **RBL-3-1433**






EXPANSION BEARING DETAIL.

Scale: 3" = 1'-0"



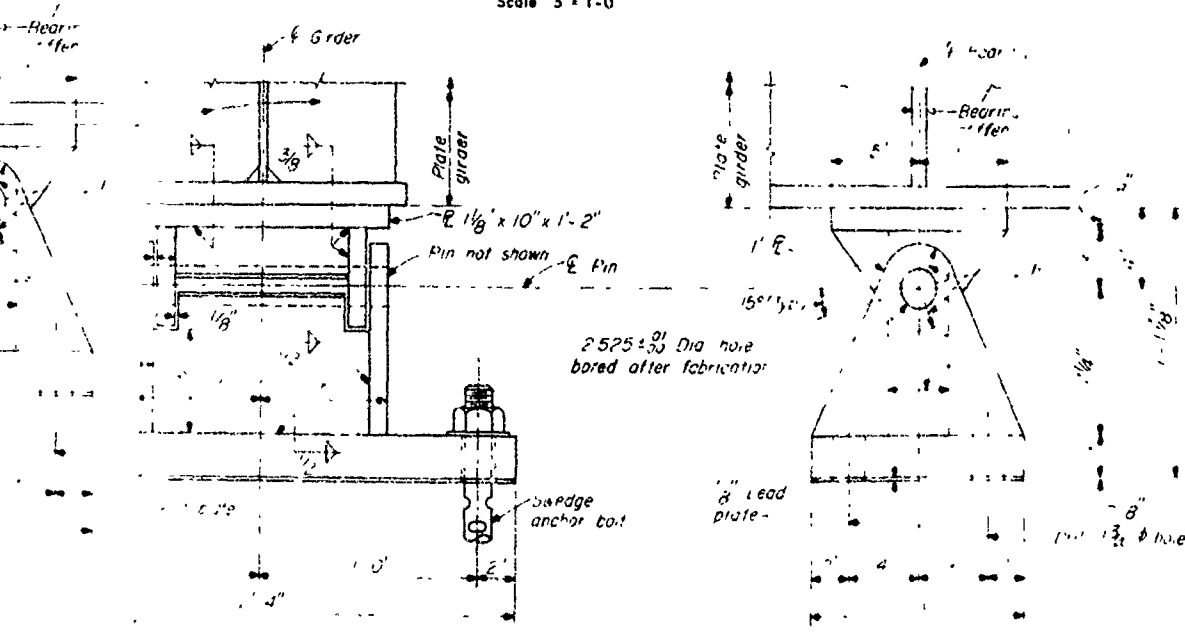
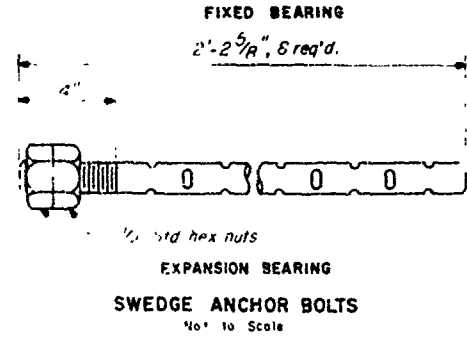
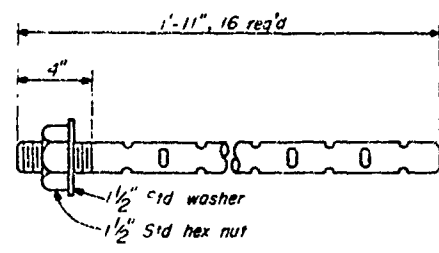
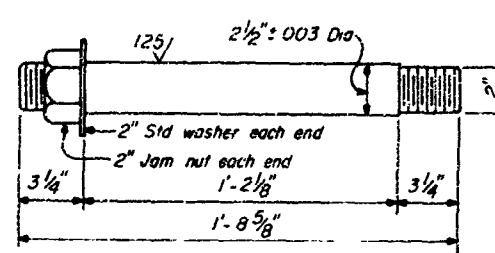
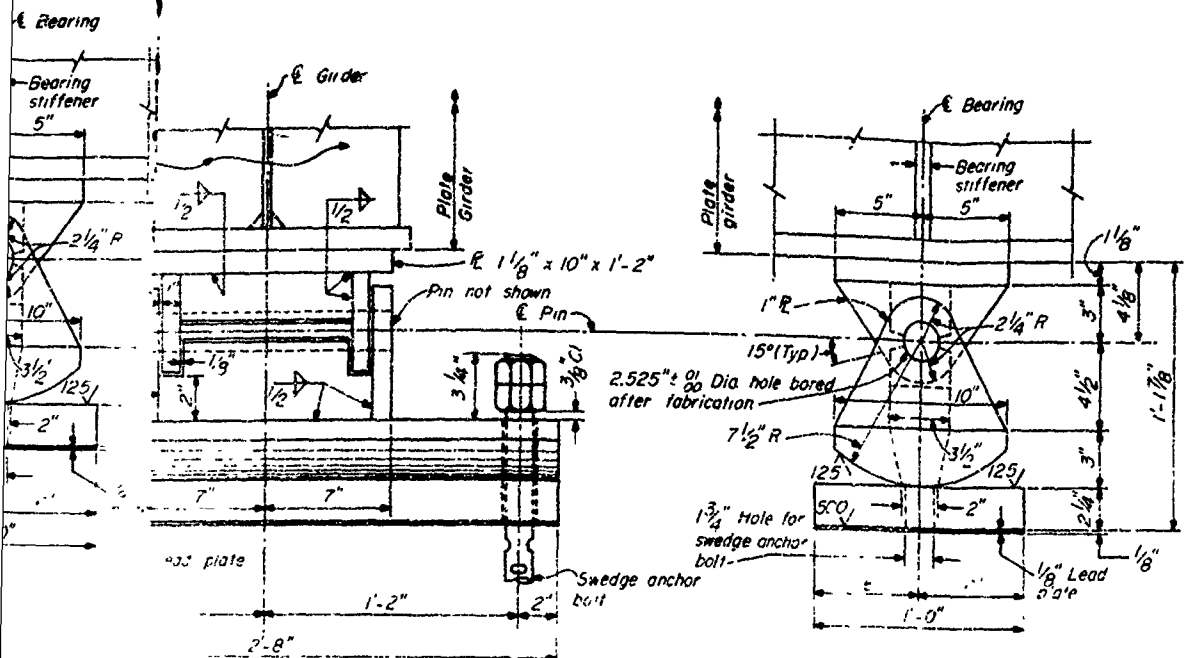
FIXED BEARING DETAIL

Scale: 3" = 1'-0"

	ROCKER AND EXPANSION R SETTING		
	INTAKE TOWER	PIER AND/OR ABUTMENT	
Temperature at time of setting			
0°F	1/2"	3 1/2"	1/2"
50°F	1/2"	3"	0
100°F	1/2"	2 1/2"	+ 1/2"

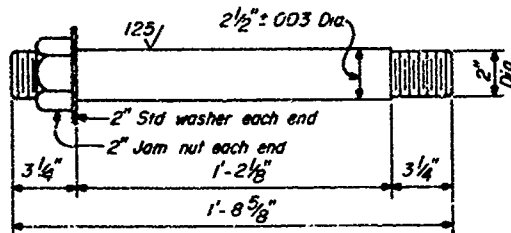
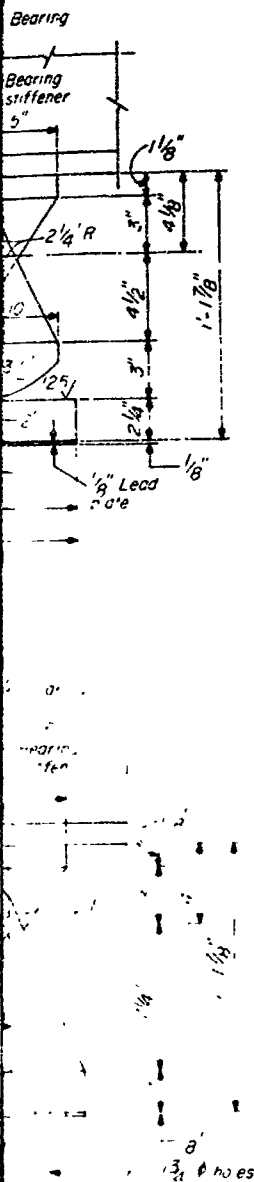
NOTE:

Axis of rocker is vertical at 50° F  
 For temperatures other than those shown, use straight line interpolation to determine proper position

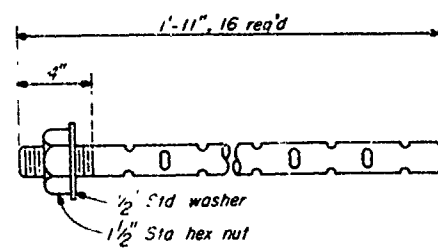


ROCKER AND EXPANSION R SETTING		
RAIL TOWER	PIER AND/OR ABUTMENT	
2	3 1/2"	1/2"
2	3"	0
1/2"	2 1/2"	+ 1/2"

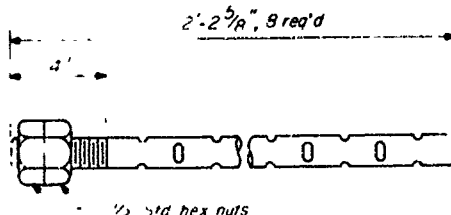
is vertical at 50° F  
other than those  
straight line interpolation  
number position



PIN  
Not to Scale



FIXED BEARING

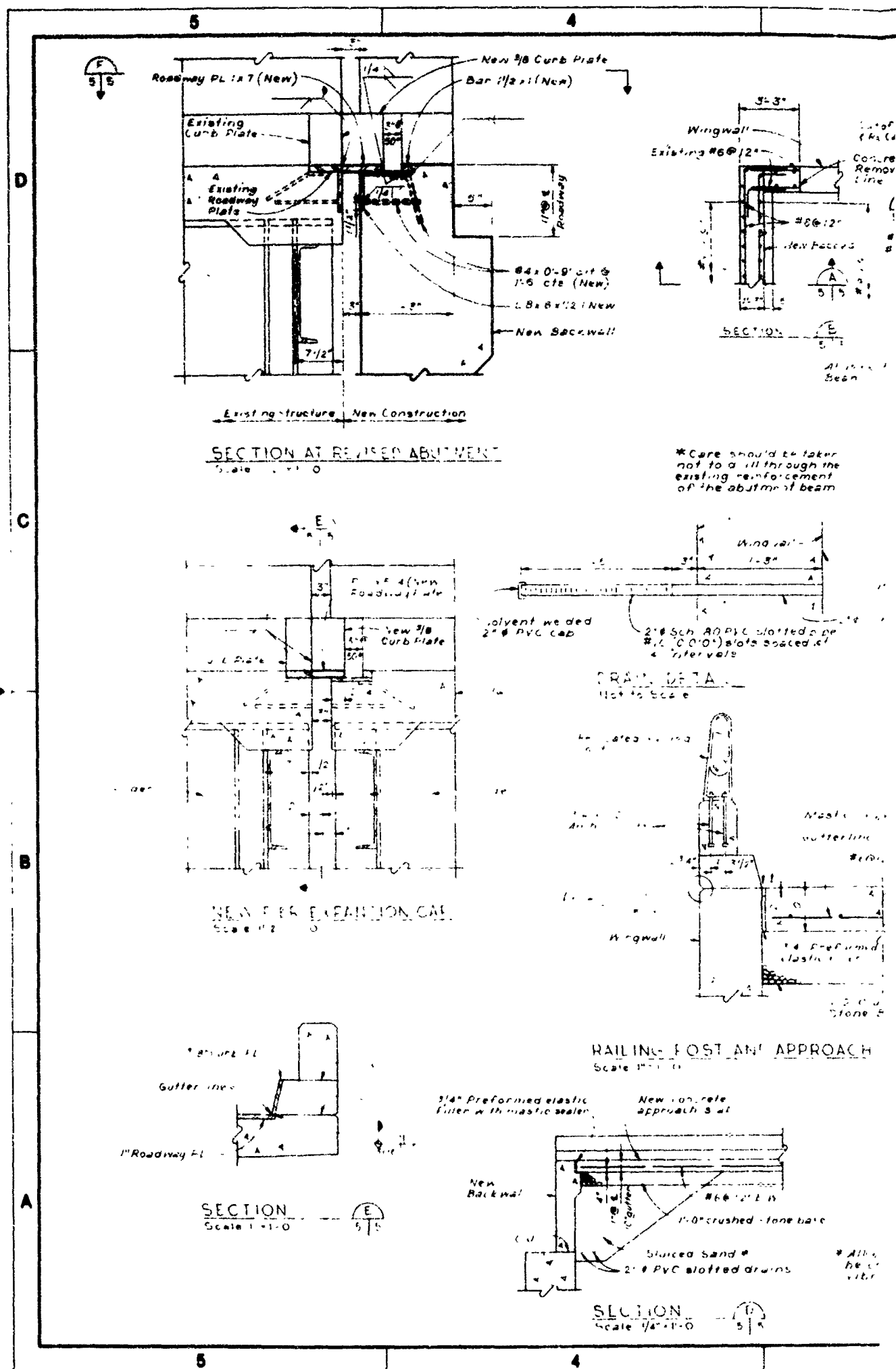


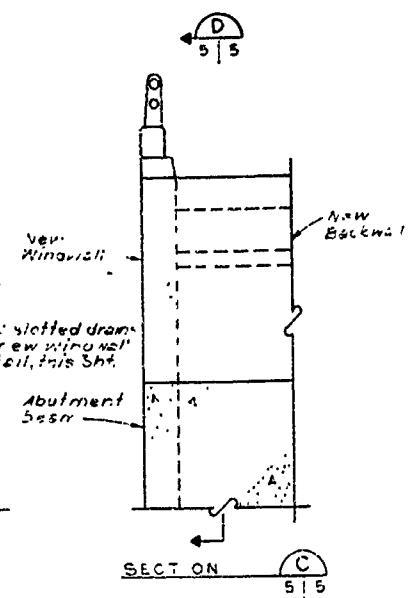
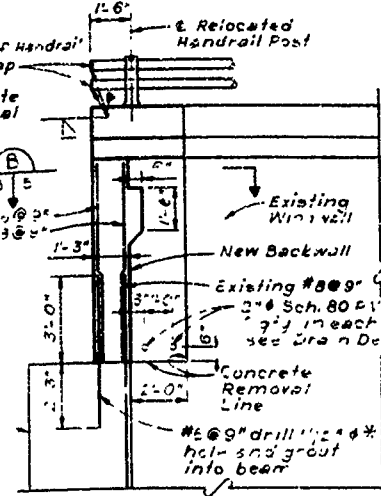
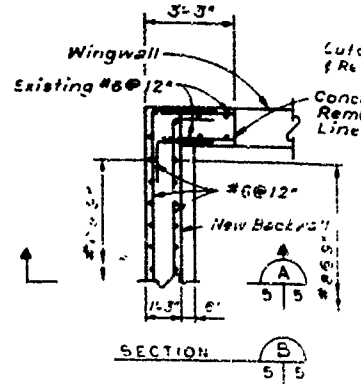
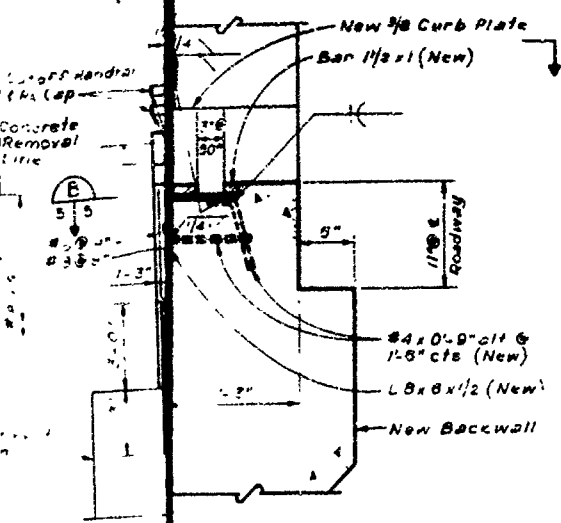
EXPANSION BEARING  
SWEDGE ANCHOR BOLTS  
Not to Scale

# NOTES

- 1 Shoes fabricated from structural steel plates. Where machined surfaces were required, sufficient stock was allowed to provide the dimensions as shown. Shoes were stress relieved by heat treating after welding.
- 2 Nuts for the anchor bolts on fixed bearings were tightened and the threads punched to lock nuts. Expansion bearings had the nuts tightened against each other.
- 3 Bolts may be deformed cold or heated to make swedge anchor bolts.
- 4 Anchor bolts for bearings are 1 1/2 inch swedge bolts and extend 18 inches into concrete.
- 5 Used a Non-Ferrous, Non-Shrinking grout.

Revisions			
Symbol	Descriptions	Date	Approved
<p>U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p>			
Designed by	RGD	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</p>	
Drawn by	P.R.C.	<p>SERVICE BRIDGE MISCELLANEOUS DETAILS</p>	
Checked by	RGD/WBE	Scale	AS SHOWN
Submitted by	RGD	Date	
		File No.	RBL-3-1434





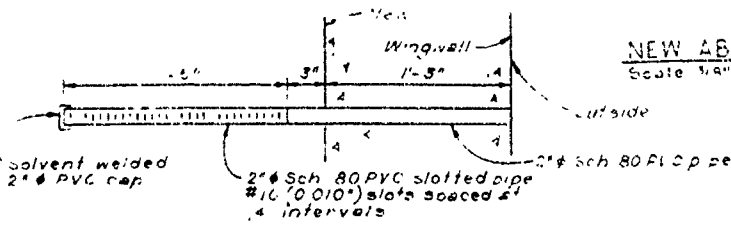
\* Care should be taken not to drill through the existing reinforcement of the abutment beam

ELEVATION A

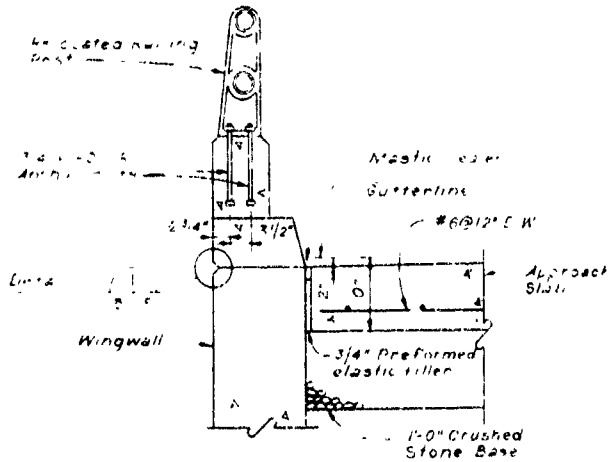
SECTION C

NEW ABUTMENT BACKWALL  
Scale 3/4" = 1'-0"

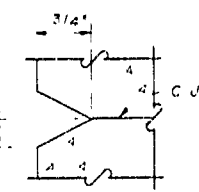
A



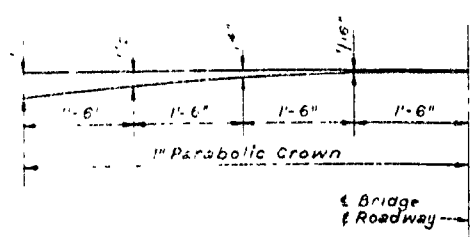
DRAIN DETAIL  
Not to Scale



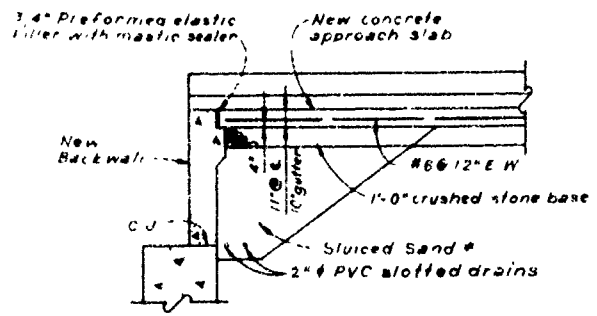
RAILING POST AND APPROACH SLAB DETAIL  
Scale 1" = 1'-0"



DETAIL (RUSTICATION)  
Full Scale



ROADWAY SLAB CROWN  
Not to Scale

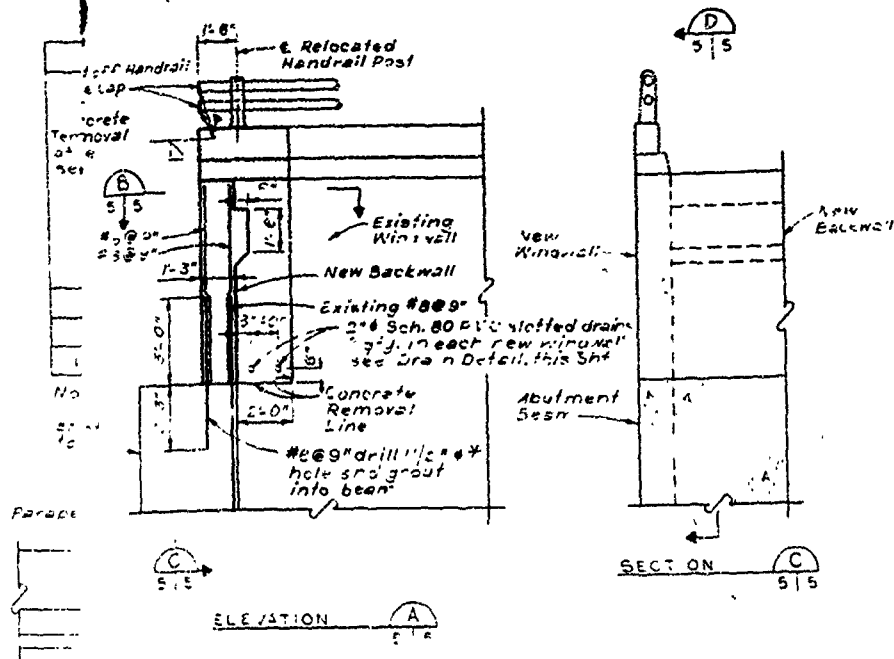


SECTION D  
Scale 1/4" = 1'-0"

\* All fill material will be compacted by plate vibrator



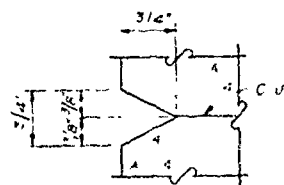




NEW ABUTMENT BACKWALL  
Scale: 1/8" = 1'-0"

SE  
Scale: 1/8" = 1'-0"

GENERAL NOTES:  
1. Concrete f'c = 3,020 psi  
2. Reinforcing steel f<sub>s</sub> = 24,000 psi  
3. Structural steel f<sub>s</sub> = 20,000 psi  
4. Clear distance of reinforcement shall be 2" unless otherwise noted  
5. Top of backwall and expansion devices to conform to the crown of the roadway slab  
6. CRES = Corrosion Resisting Steel  
7. C.U. = Construction Joint  
8. Curb and roadway plates will be preheated before welding  
9. All exposed concrete shall be chamfered 3/4"



DETAIL  
(RUSTICATION)  
Full Scale

2'-5" W

At road  
slab

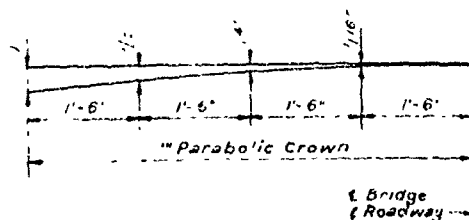
1'-6" base

SLAB DETAIL

Symbol	Change
(A)	Revise

Designed by: RCL  
Drawn by: PWM  
Checked by: RCL  
Submitted by: TDW

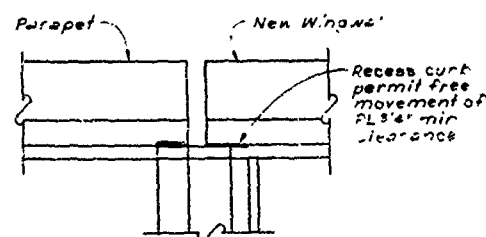
Material will  
be impacted by plate



ROADWAY SLAB CROWN  
Not to scale

PIER AND/OR ABUTMENT	
Temperature at time of setting	
0°F	3 1/2"
50°F	3"
100°F	2 1/2"

Note:  
For temperatures other than those shown, use straight line interpolation to determine proper dimension.



SECTION  
Scale: 1/8" = 1'-0"

#### GENERAL NOTES

- Concrete f'c = 3,020 psi
- Reinforcing steel f<sub>s</sub> = 24,000 psi
- Structural steel f<sub>s</sub> = 20,000 psi
- Clear distance of reinforcement shall be 2" unless otherwise noted
- Top of backwall and expansion devices to conform to the crown of the roadway slab
- CRES = Corrosion Resisting Steel
- C.U. = Construction Joint
- Curb and roadway plates will be preheated before welding
- All exposed concrete shall be chamfered 3/4"

## RECORD DRAWING

JANUARY 1990  
CONTRACT NO. DACW1-88-C-0181

Symbol	Revisions	Date	Approved
(A)	Change size of rebar	6-21-89	JCL
	Revised for "As Built" conditions	3-13-90	TDW

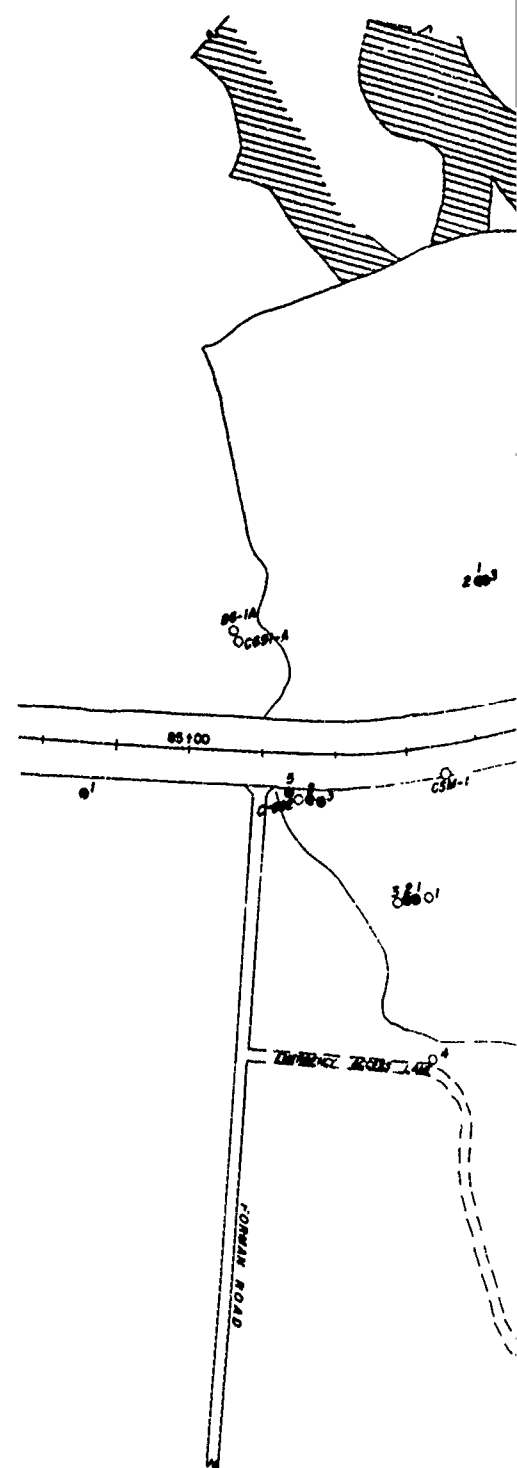
U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: RCL	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT		
Drawn by: PWM	SERVICE BRIDGE ALTERATION		
Checked by: RCL	Scale: AS SHOWN	Sheet number:	
Submitted by: T.D.W.	Date: MAY 1989	File No.:	RBL-3-1701

PLATE NO. 34A

## OBSERVATION DEVICE SCHEDULE

DEVICE	STATION	RANGE	UP ELEVATION	TOP ELEVATION	INSTALLATION	DETAILS
PPF 05-1	85+00	0+00	927.4	805.8	FDN. SHALE	1
PPF 05-1A	85+00	0+00	929.2	783.5	FDN. SHALE	1
PPF 07	96+19	2+00	688.7	773.1	FDN. CL.	1
PPF 07-2	86+70	0+00	936.5	754.5	FDN. SANDSTONE	1
PPF 07-3	86+85	0+00	928.5	766.5	FDN. SILTSTONE	1
PPF 07-4	87+10	2+00	691.4	763.4	FDN. ROCK STRATA	70
PPF 07-5	86+40	0+00	927.5	859.5	FDN. ROCK STRATA	70
PPF 08-1	87+99	1+00	903.0	837.4	FDN. ROCK STRATA	70
PPF 08-2	87+88	1+00	902.9	837.4	FDN. ROCK STRATA	70
PFE 08-3	87+77	1+00	902.4	879.9	SAND DRAIN	1
PZF 229	91+00	0+00	919.1	809.9	FDN. CC	1
PPE 09-1	89+80	1+00	903.1	834.9	EMB. CL.	1
PPE 09-2	89+45	1+00	902.1	861.8	EMB. CH	1
PPE 09-3	89+88	1+00	902.9	818.0	EMB. CL.	1
PZE 09-9	89+13	1+00	888.0	819.8	SAND DRAIN	1
PZE 09-2	90+81	2+00	849.6	823.9	SAND DRAIN	1
PPE 02-1	92+00	1+00	908.3	823.0	EMB. CL.	1
PPE 02-2	92+00	1+10	904.9	823.3	EMB. CL.	2
PPE 03-1	93+00	3+00	818.9	818.9	EMB. CH	3
PPF 03-2	93+00	0+00	921.3	811.4	FDN. CH	1
PPF 03-2A	93+00	1+00	---	775.0	FDN. SILTSTONE	3
PPE 03-4	93+00	0+00	924.0	819.5	EMB. CH	1
PPE 03-4A	93+00	2+00	897.2	819.4	EMB. CH	1
PPE 03-5A	93+00	0+10	---	812.0	EMB. CL.	3
PPE 03-5A	93+00	0+50	---	830.0	EMB. CH	3
PPF 03-8	93+00	0+50	923.3	785.0	FDN. SILTSTONE	1
PZF 03-9	93+00	1+00	900.4	864.9	FDN. CC	1
PZF 03-10A	93+00	3+00	867.2	805.9	FDN. CLAYEY SAND AND GRAVEL	1
PZF 03-11	93+00	5+30	820.4	803.7	FDN. CLAYEY GRAVEL AND SAND	1
PPE 03-12	93+00	1+10	---	830.0	EMB. CH	3
PPE 06-1	96+00	1+00	---	820.0	EMB. CH	3
PPE 06-1	96+70	1+00	907.2	826.4	EMB. CL.	1
PZE 06-2	96+70	3+00	846.6	796.3	SAND DRAIN	1
PPE 07-1	97+00	3+25	---	820.3	EMB. CL. SEWER BACKFILL	3
PPF 07-2	97+00	2+10	---	820.6	EMB. CL.	1
PPF 07-3	97+00	1+00	---	---	EMB. CL.	1
PPF 07-4	97+00	1+00	---	---	EMB. CL.	1
PPF 07-5	97+00	1+00	---	---	EMB. CL.	1
PPF 07-6	97+00	1+00	---	---	FDN. SHALE	3
PZF 07-10	97+00	1+00	899.2	802.8	FDN. CC	1
PZE 07-11	97+00	1+00	890.9	824.4	SAND DRAIN	1
PPE 07-13	97+32	1+00	901.8	839.7	EMB. CL.	1
PPE 07-14	97+36	1+00	902.2	830.0	EMB. CL.	1
PPE 07-15	97+35	2+00	---	829.1	EMB. CL.	5
PPE 07-16	97+00	0+00	916.8	840.0	EMB. CL.	1
PPE 07-17	97+00	0+75	---	838.0	EMB. CL.	3
PPE 08-1	97+80	1+00	---	829.4	EMB. CL.	3
PPE 08-2	98+00	1+00	---	830.0	EMB. CL.	3
PPF 09-1	98+40	2+00	---	800.0	FDN. SHALE	3
PPE 09-2	98+00	0+00	913.6	811.8	EMB. CL.	1
PPF 09-3	98+00	0+00	923.6	789.7	FDN. SHALE AND SILTSTONE	1
PPF 09-4	98+00	0+00	923.4	771.3	FDN. SHALE AND SILTSTONE	1
PPE 09-5	98+00	0+00	---	818.0	EMB. CL.	1
PPE 09-6	99+00	2+00	---	822.5	EMB. CL.	2
PPE 09-7	98+00	0+70	916.8	804.5	EMB. CL. FDN. SHALE CONTACT	1
PPE 09-8	98+45	0+00	---	836.7	EMB. CL.	5
PPE 09-10	99+00	1+00	891.8	823.1	EMB. CL.	1
PPE 09-11	98+44	0+00	---	827.4	EMB. CL.	6
PPE 09-12	98+40	---	---	839.0	EMB. CL.	5
PPE 09-13	98+00	2+00	896.9	849.2	EMB. CL.	1
PPE 09-16	99+10	0+00	---	836.4	EMB. CL.	3
PPE 09-17	98+80	1+00	---	830.8	EMB. CL.	3
PPE 09-18	98+83	0+00	---	801.5	EMB. CH FDN. SANDSTONE CONTACT	6
PPF-C691A	85+68	1+00	929.7	803.1	FDN. ROCK STRATA	6
PPF-C692	86+90	0+00	926.7	799.4	FDN. ROCK STRATA	6
PPF-C693	103+00	1+00	914.0	811.1	FDN. ROCK STRATA	6
PPF-C694	103+00	1+00	913.2	847.1	FDN. ROCK STRATA	6
M-1	88+04	8+50	859.4	811.6	FDN. OVERBURDEN	70
M-2	87+33	13+00	865.2	880.3	FDN. OVERBURDEN	70
M-3	89+31	13+00	882.9	808.0	FDN. OVERBURDEN	70
M-4	91+39	12+00	826.9	808.8	FDN. OVERBURDEN	70
M-5	91+68	10+00	827.9	---	FDN. OVERBURDEN	70

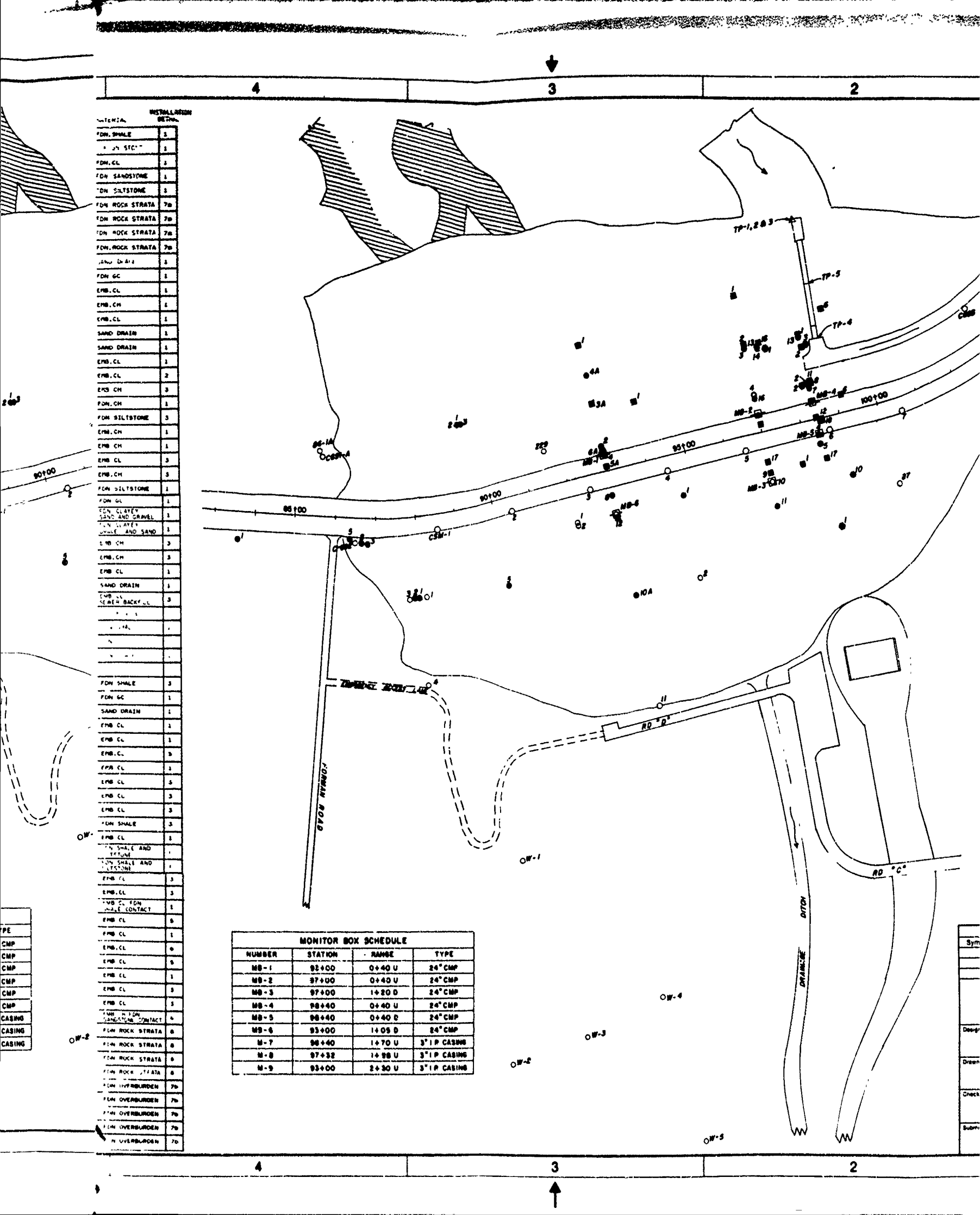


## MONITOR BOX SCHEDULE

NUMBER	STATION	RANGE	TYPE
MB-1	93+00	0+40 U	24" CMP
MB-2	97+00	0+40 U	24" CMP
MB-3	97+00	1+20 D	24" CMP
MB-4	98+40	0+40 U	24" CMP
MB-5	98+40	0+40 D	24" CMP
MB-6	93+00	1+05 D	24" CMP
M-7	96+40	1+70 U	3" I.P. CASING
M-8	97+32	1+98 U	3" I.P. CASING
M-9	93+00	2+30 U	3" I.P. CASING

TYPE
CMP
CMP
CMP
CMP
CMP
CMP
CASING
CASING
CASING

MONITOR BOX SCHEDULE			
NUMBER	STATION	RANGE	TYPE
MB-1	92+00	0+40 U	24" CMP
MB-2	37+00	0+40 U	24" CMP
MB-3	97+00	1+20 D	24" CMP
MB-4	98+40	0+40 U	24" CMP
MB-5	98+40	0+40 D	24" CMP
MB-6	93+00	1+05 D	24" CMP
M-7	96+40	1+70 U	3" I P CASING
M-8	97+32	1+88 U	3" I P CASING
M-9	93+00	2+30 U	3" I P CASING



3

2

1

TP-1, 2 &amp; 3

TP-5

TP-4

100+00

95+00

90+00

10A

RD "B"

DITCH

DRAINAGE

OW-4

OW-3

OW-2

OW-5

WW

WW

- LEGEND
- PZ OPEN TUBE PIEZOMETER
  - VW VIBRATING WIRE PIEZOMETER
  - PP PRESSURE CELL PORE PRESSURE DEVICE
  - AP OPEN TUBE PORE PRESSURE DEVICE
  - F FOUNDATION
  - E EMBANKMENT
  - SP SETTLEMENT MONITORING DEVICE
  - AL ALIGNMENT MONUMENT
  - LS LEST SETTLEMENT MONUMENT
  - IM IMMONUMENT MONUMENT
  - PH MONITOR BOX
  - I IN INCH
  - TP TPA

DETAIL A - IN DETAIL LEGEND

DETAIL NO.	PIEZOMETER TYPE
1	3/4" OPEN TUBE PIEZOMETER
2	2" OPEN TUBE PIEZOMETER
3	PNEUMATIC PIEZOMETER
4	VIBRATING WIRE PIEZOMETER
5	PNEUMATIC CELL INSTALLED IN EXISTING 1" OPEN TUBE PIEZOMETER
6	PNEUMATIC CELL INSTALLED IN EXISTING 1" OPEN TUBE PIEZOMETER
7a	3/4" OPEN TUBE PIEZOMETER INSTALLING SEVERAL STRATA 1 1/2" x 24" 820 TO 020" SLOTTED TIP
7b	2" OPEN TUBE PIEZOMETER INSTALLING SEVERAL STRATA 2" x 16" 820 TO 020" SLOTTED TIP
8	3/4" OPEN TUBE PIEZOMETER INSTALLING SEVERAL STRATA 1 1/2" x 24" 820 TO 020" SLOTTED TIP

Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: RGO

DRAWN BY: R A H

CHECKED BY: RGO/WEE

SUBMITTED BY: RGO

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

OBSERVATION DEVICES  
PLAN AND SCHEDULE

Scale: AS SHOWN

Date:  

Sheet number:  

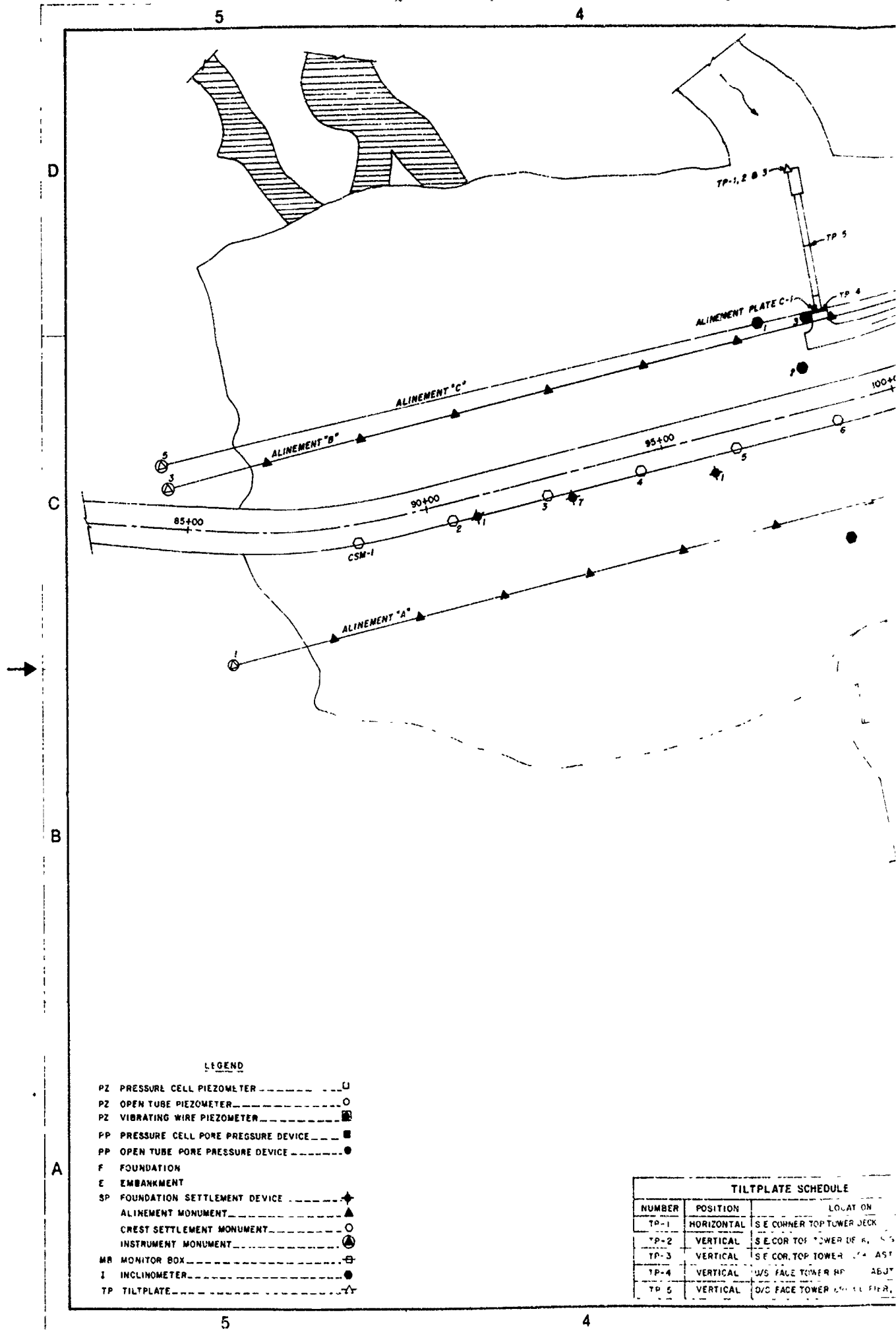
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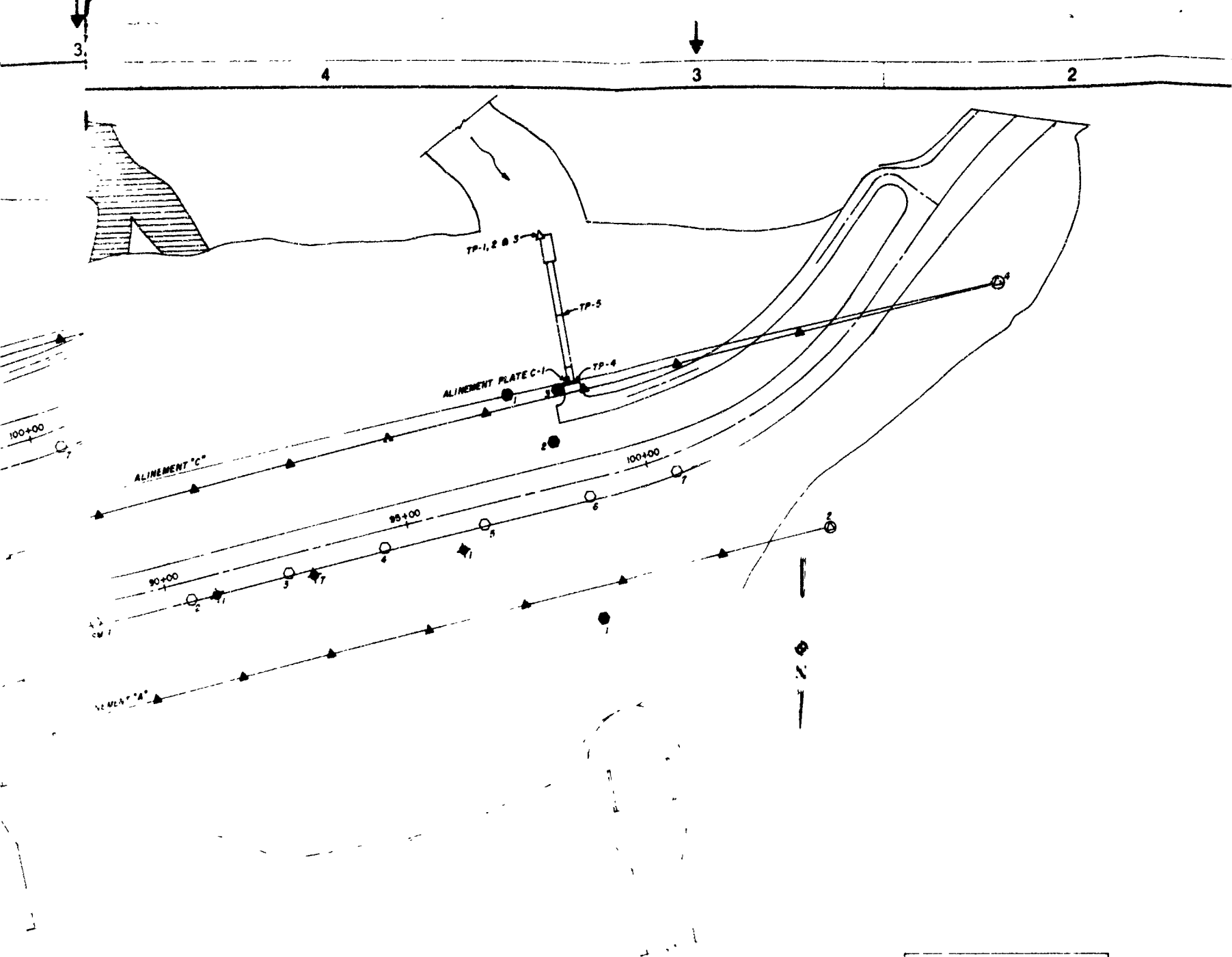
PLATE NO 35

3

2

1





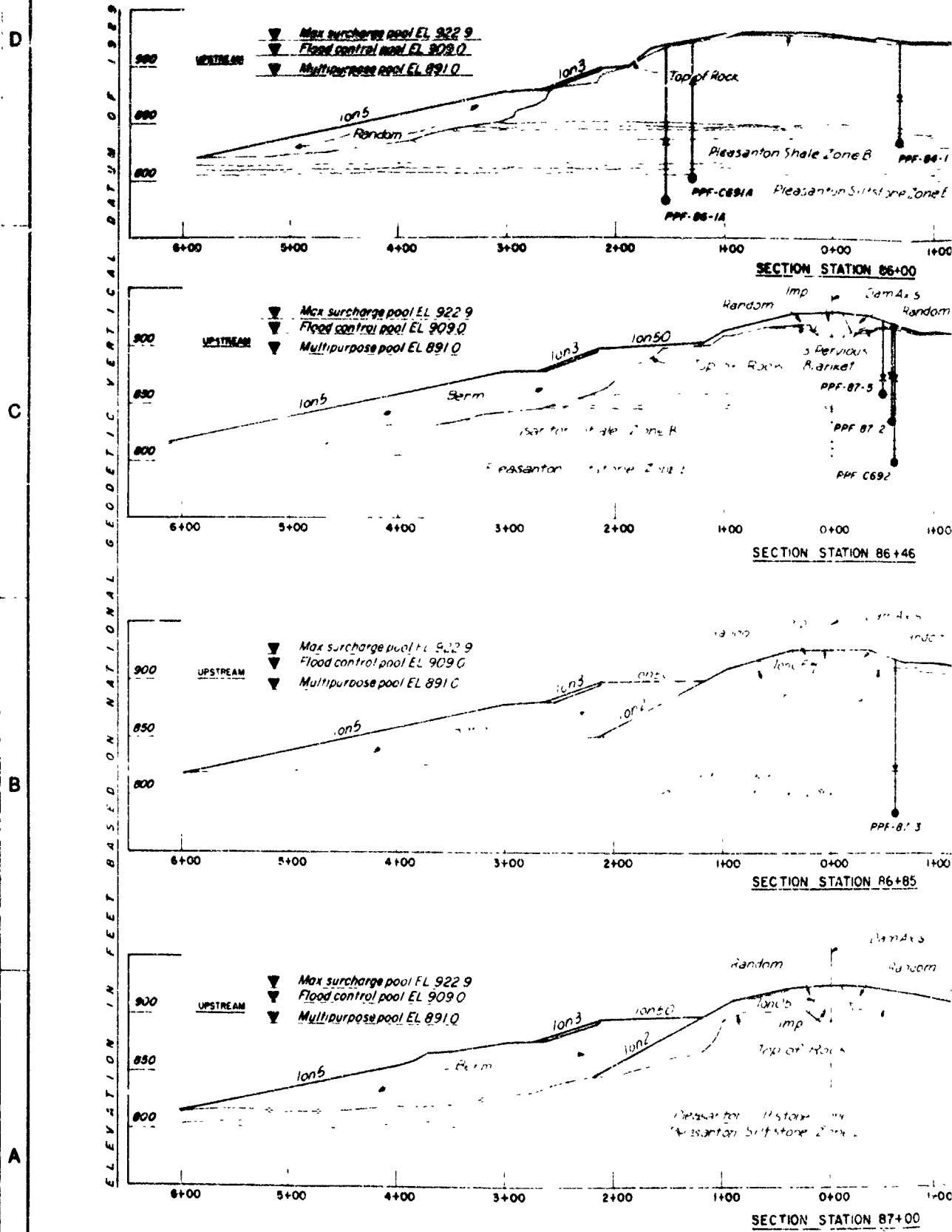
SCHEDULE OF FOUNDATION SETTLEMENT DEVICES		
NO	STATION	RANGE
SPPF-91-1	91+00	0+40 D
SPPF-93-7	93+00	0+50 D
SPPF-96-1	96+10	0+75 D

INCLINOMETER SCHEDULE		
NUMBER	STATION	RANGE
I 99-1	98+40	2+57 D
I 99-2	98+25	0+85 U
I 99-3	98+50	1+80 U
I 97-1	97+49	1+89 U

TILTPLATE SCHEDULE		
NUMBER	POSITION	LOCATION
TP-1	HORIZONTAL	SE CORNER TOP TOWER DECK
TP-2	VERTICAL	SE COR TOP TOWER DECK, S GUARDRAIL
TP-3	VERTICAL	SE COR TOP TOWER DECK, S GUARDRAIL
TP-4	VERTICAL	1/2 FACE TOWER BR G ABUT EL 890.5
TP-5	VERTICAL	1/2 FACE TOWER BRIDGE PIER, EL 886.5

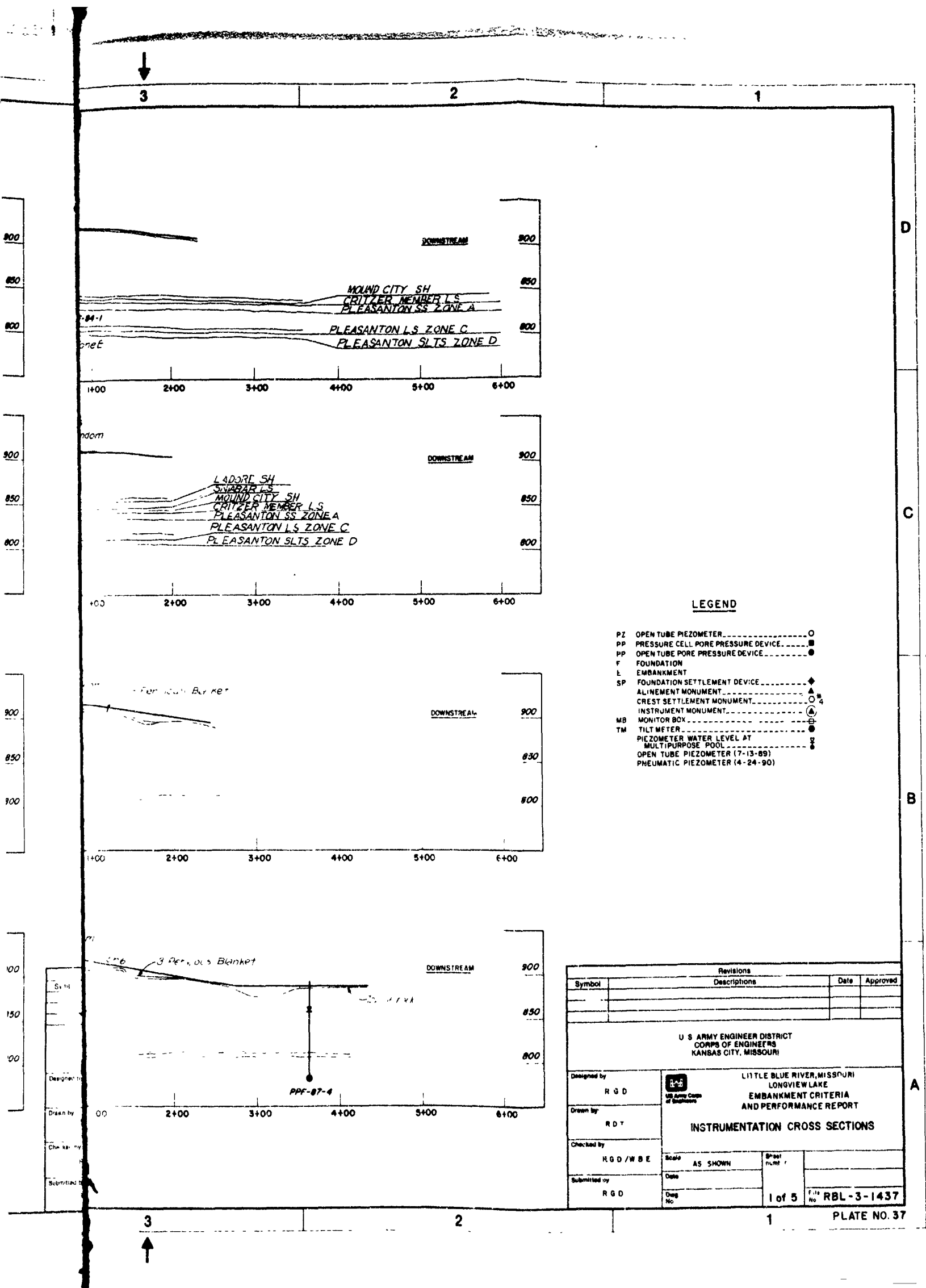
CREST SETTLEMENT MONUMENT SCHEDULE		
NUMBER	STATION	RANGE
CSM-1	88+50	0+36 D
CSM-2	90+50	0+36 D
CSM-3	92+50	0+36 D
CSM-4	94+50	0+36 D
CSM-5	96+50	0+36 D
CSM-6	98+70	0+36 D
CSM-7	100+50	0+36 D







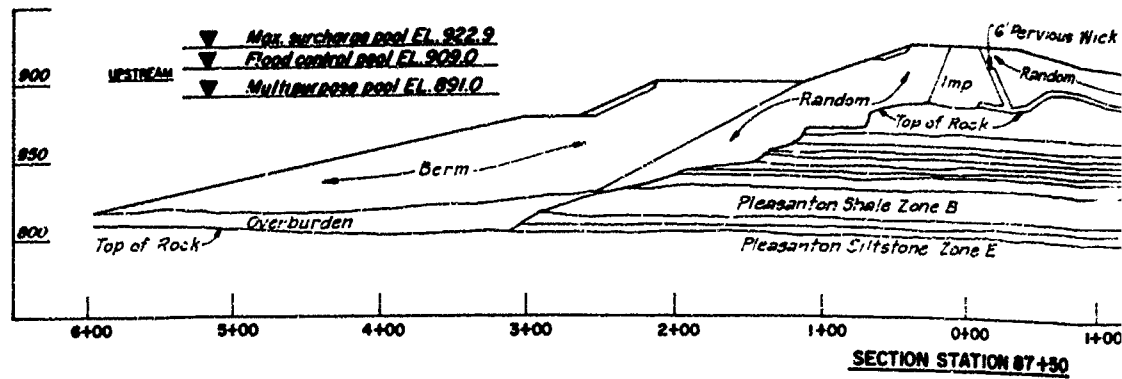






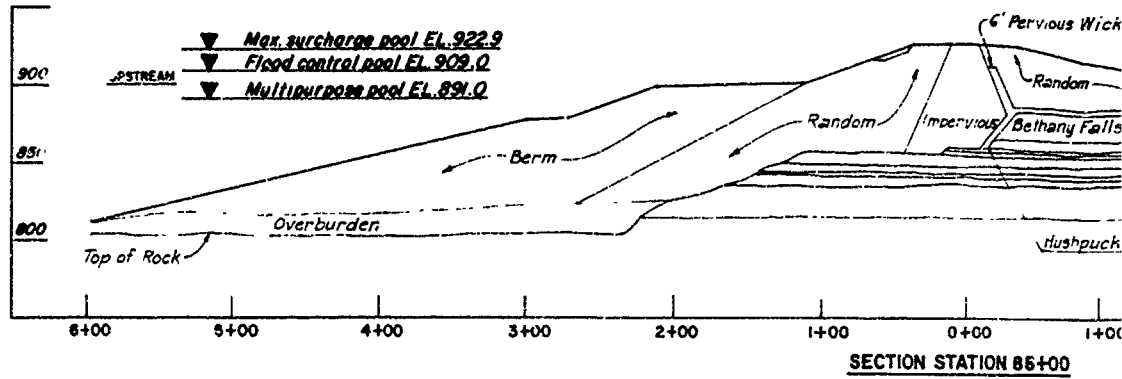
ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929

D



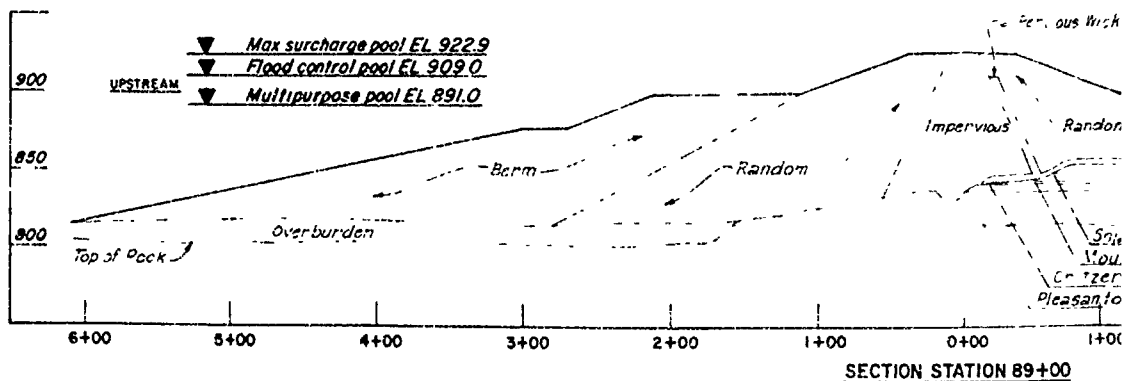
SECTION STATION 87+50

C



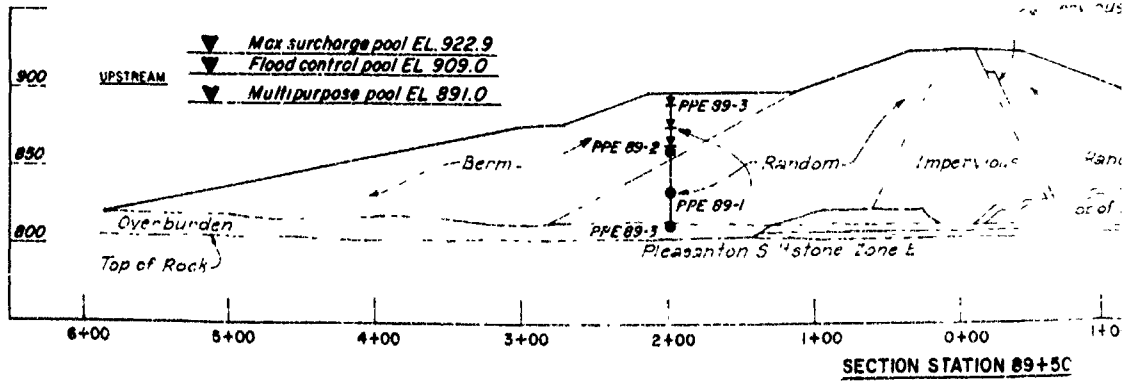
SECTION STATION 86+00

B



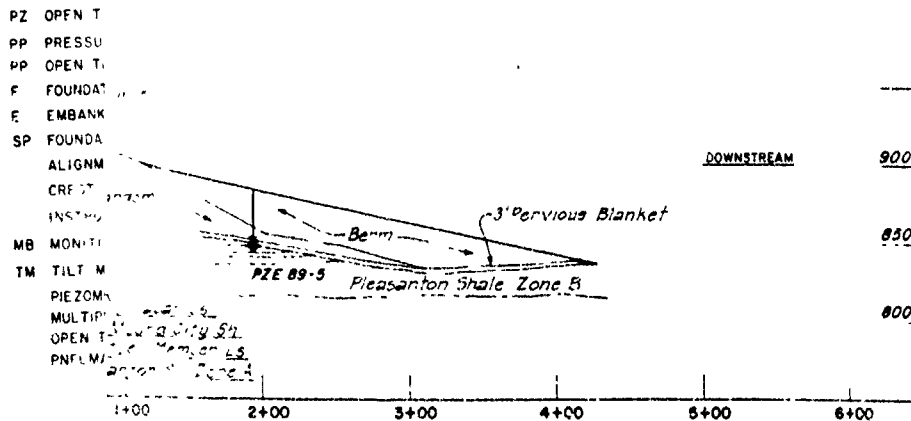
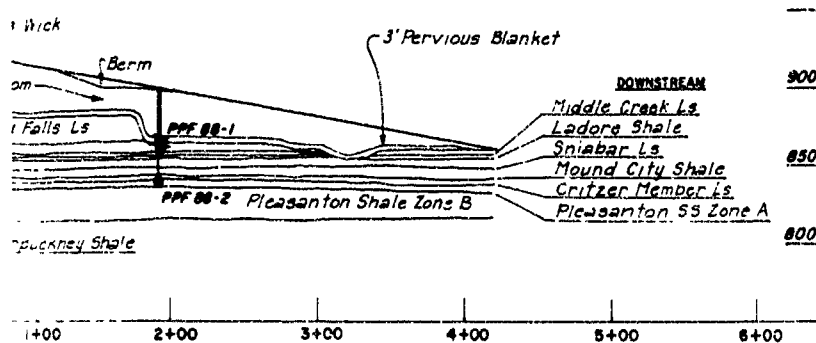
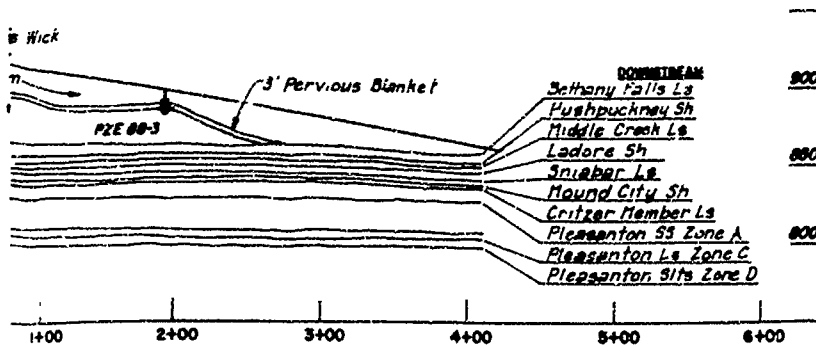
SECTION STATION 89+00

A



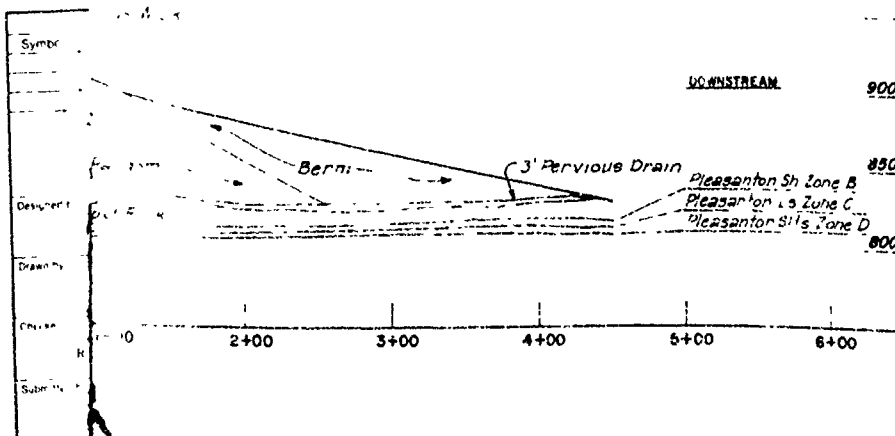
SECTION STATION 89+50





# LEGEND

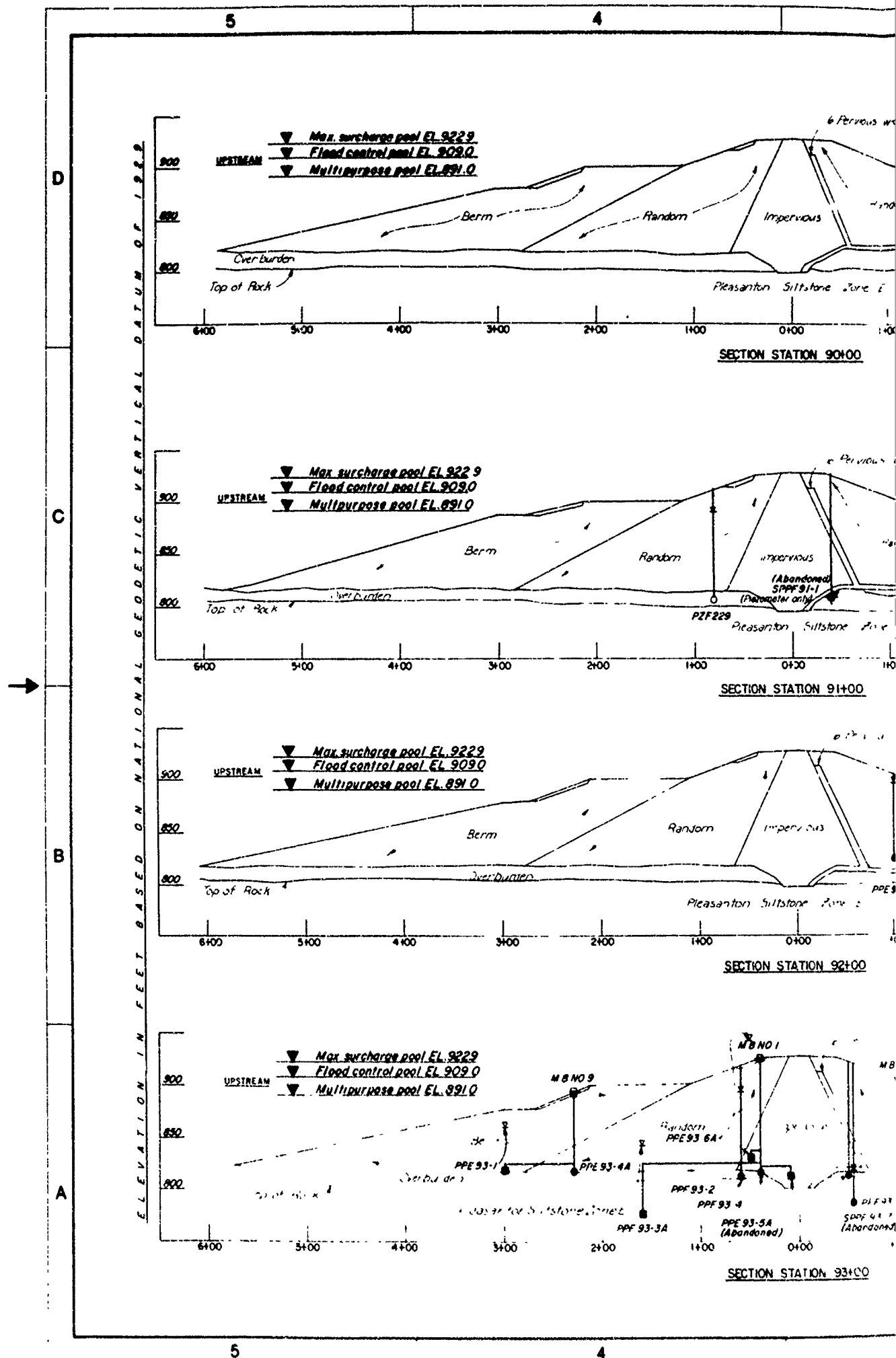
- PZ OPEN TUBE PIEZOMETER.....○
- PP PRESSURE CELL PORE PRESSURE DEVICE.....■
- PP OPEN TUBE PORE PRESSURE DEVICE.....◆
- F FOUNDATION.....
- E EMBANKMENT.....
- SP FOUNDATION SETTLEMENT DEVICE.....●
- ALIGNMENT MONUMENT.....▲
- CREST SETTLEMENT MONUMENT.....○ #4
- INSTRUMENT MONUMENT.....⊙
- MB MONITOR BOX.....⊞
- TM TILT METER.....●
- PIEZOMETER WATER LEVEL AT.....
- MULTIPURPOSE POOL.....
- OPEN TUBE PIEZOMETER (7-13-89).....
- PNEUMATIC PIEZOMETER (4-24-90).....

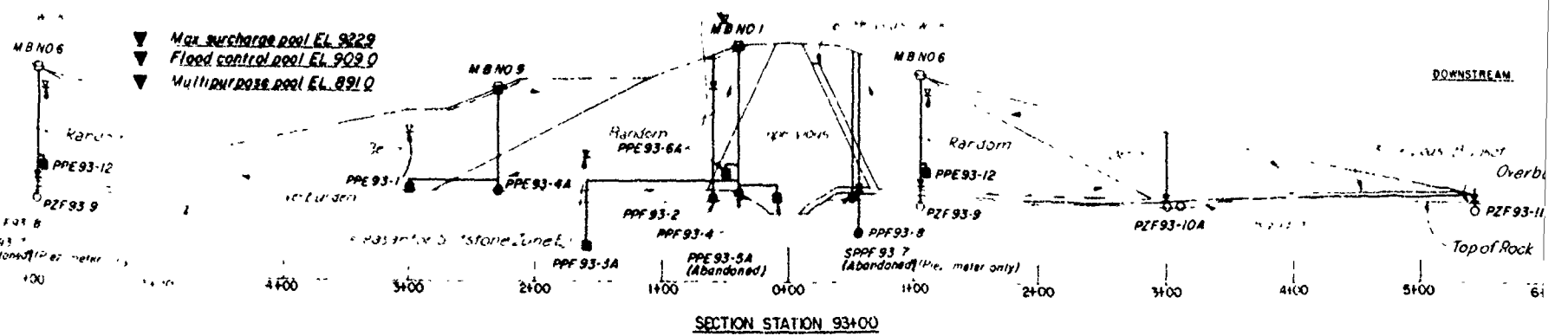
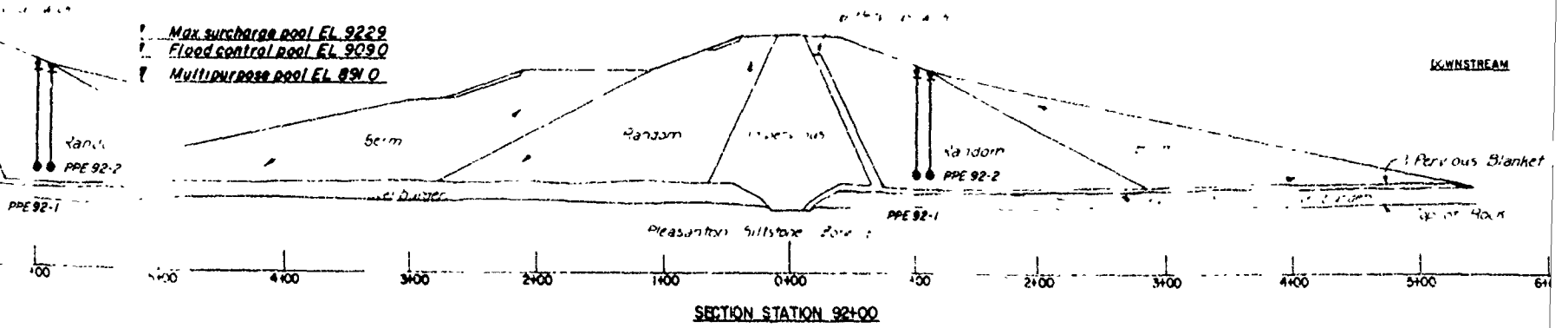
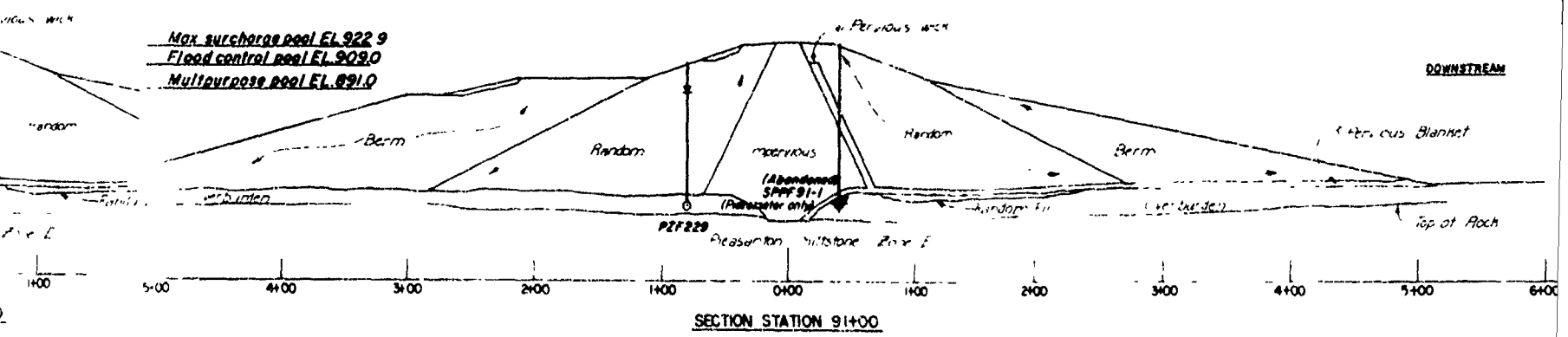
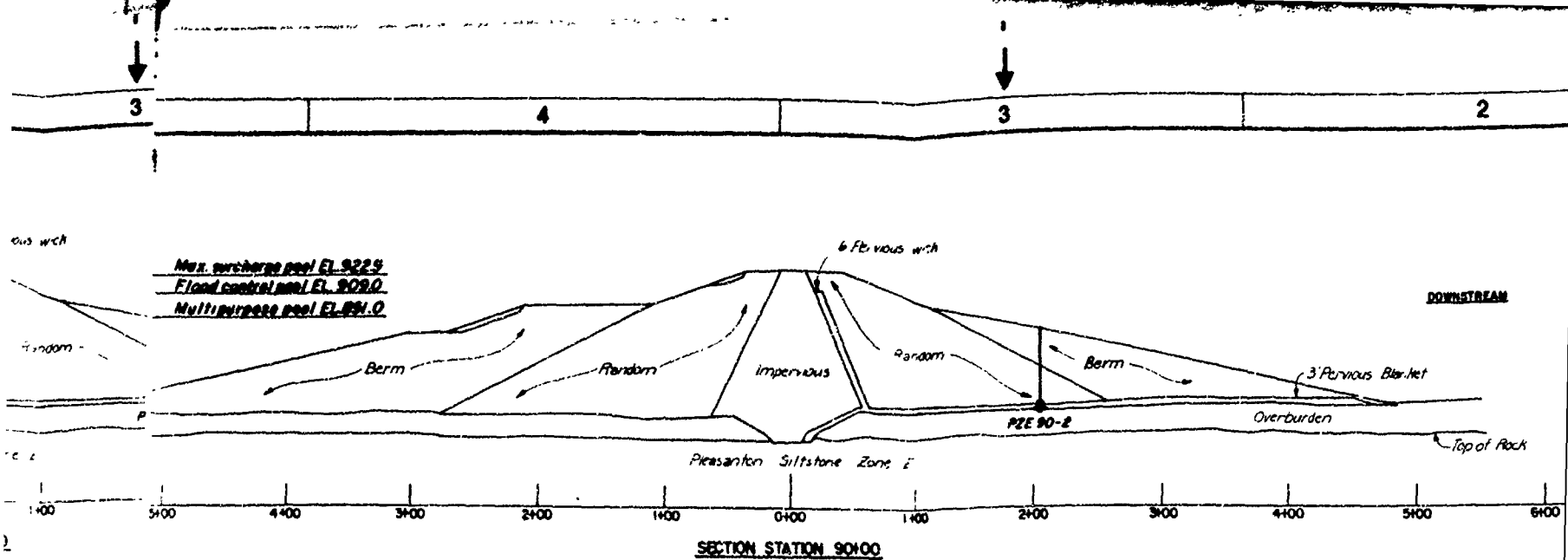


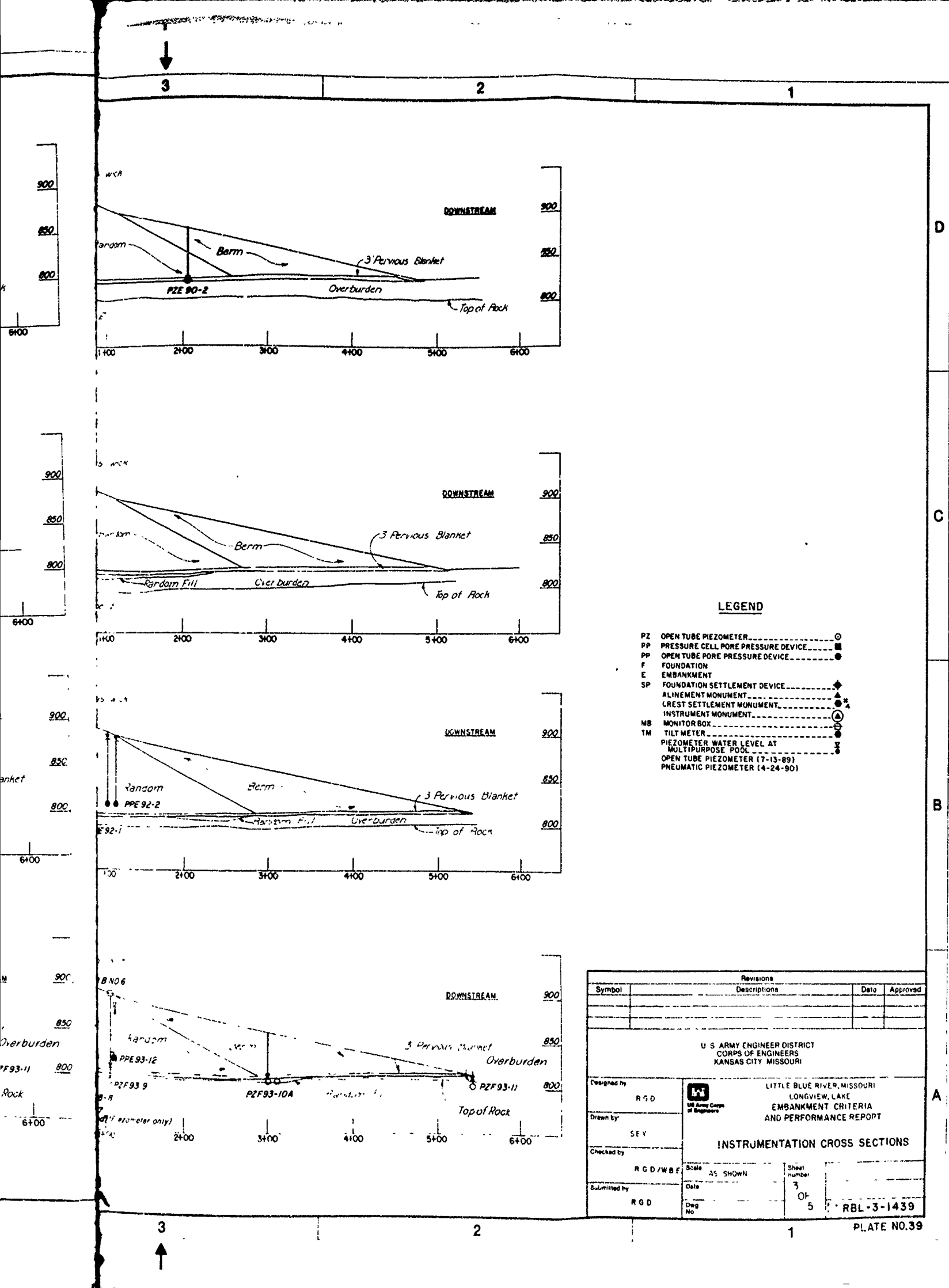
Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by R G D	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</p>	<p>INSTRUMENTATION CROSS SECTIONS</p>
Drawn by P S T		
Checked by R U D / W B E		
Submitted by R G D		
Scale AS SHOWN	Sheet number 2 OF 5	File No RBL-3-1438







Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY MISSOURI

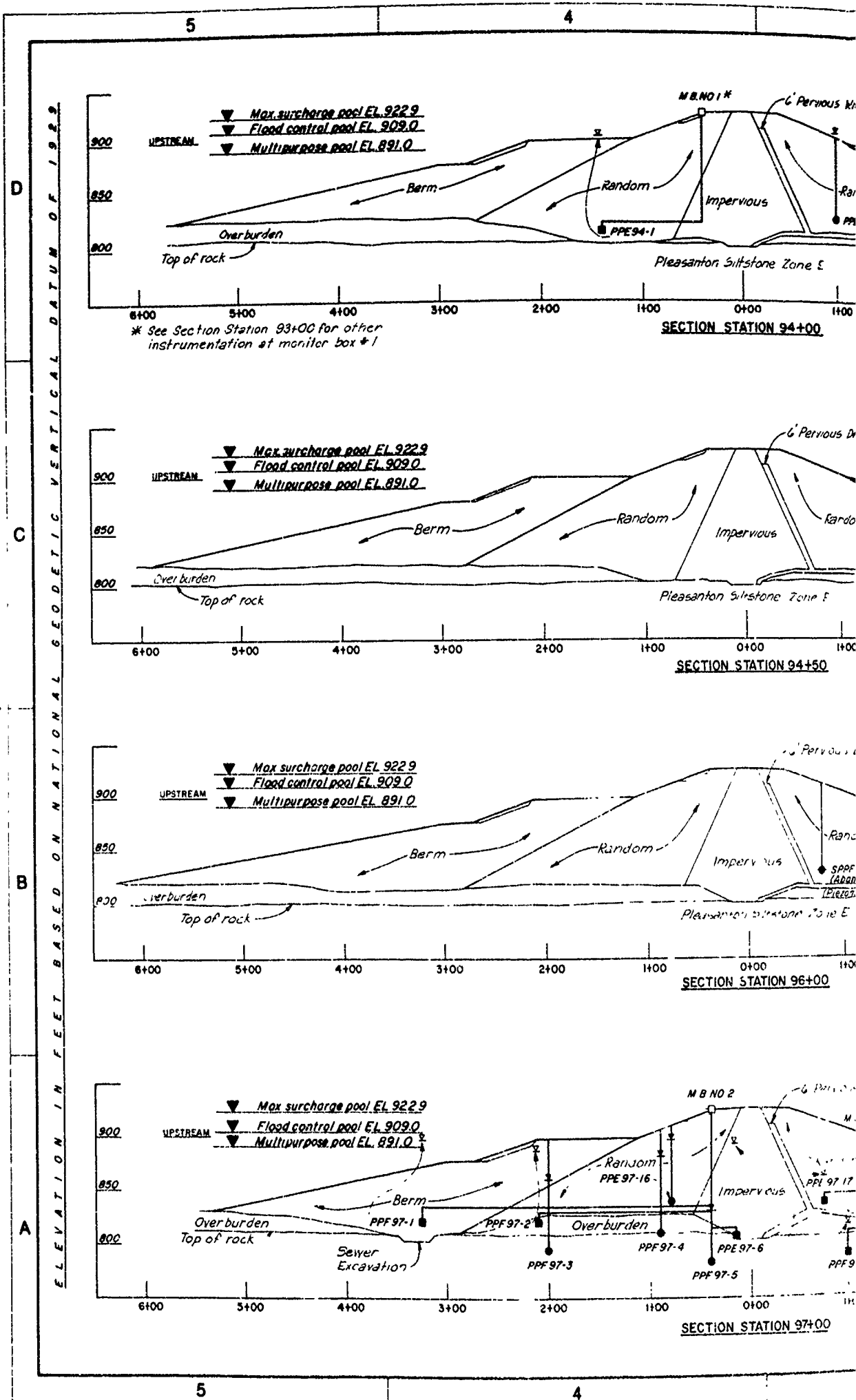
DESIGNED BY: RGD  
DRAWN BY: SEV  
CHECKED BY: RGD/WBF  
SUBMITTED BY: RGD

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW, LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

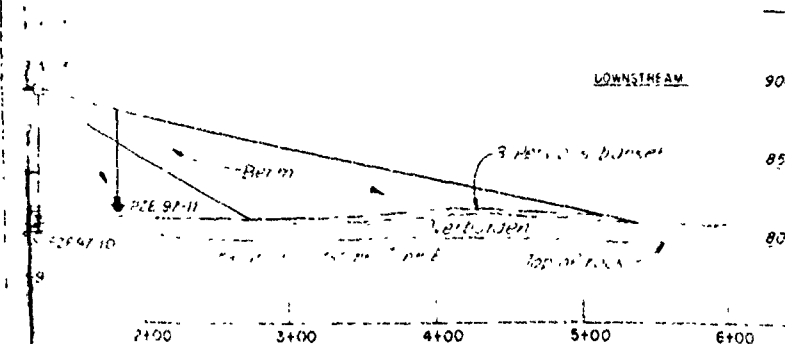
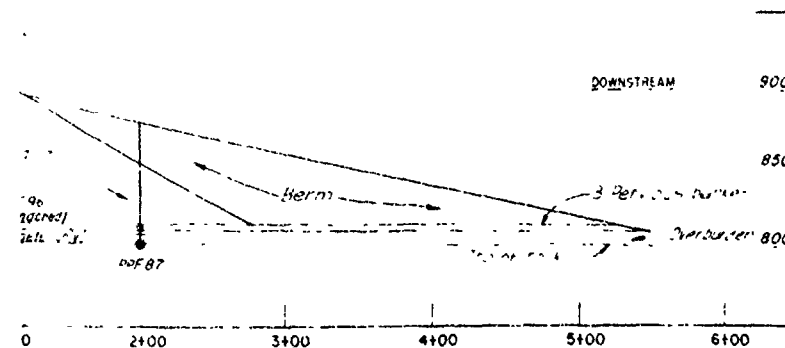
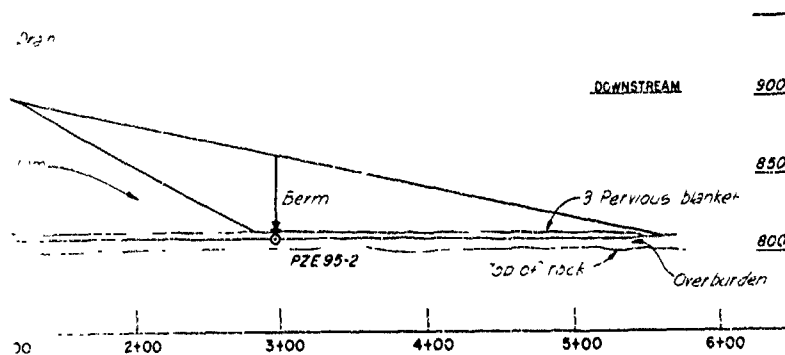
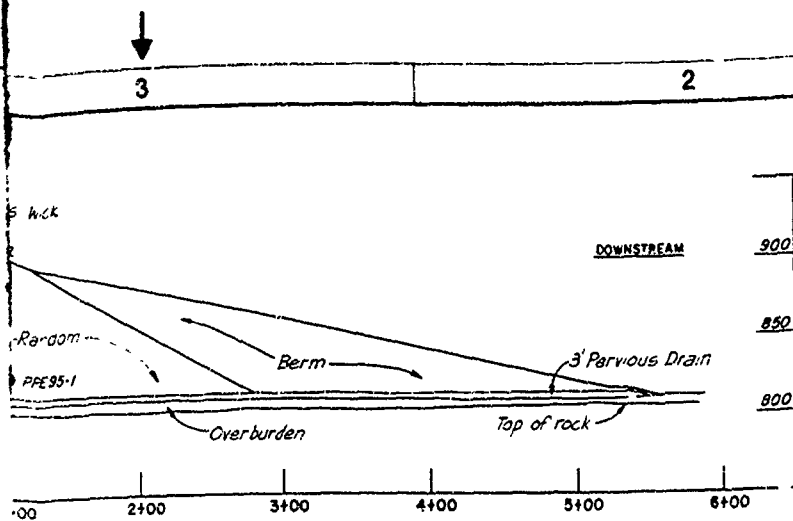
**INSTRUMENTATION CROSS SECTIONS**

Scale: AS SHOWN  
Date: 3 OF 5  
Draw No: RBL-3-1439









# LEGEND

PZ	OPEN TUBE PIEZOMETER	○
PP	PRESSURE CELL PORE PRESSURE DEVICE	■
PP	OPEN TUBE PORE PRESSURE DEVICE	●
F	FOUNDATION	—
E	EMBANKMENT	—
SP	FOUNDATION SETTLEMENT DEVICE	▲
	ALIGNMENT MONUMENT	○
	CREST SETTLEMENT MONUMENT	○
	INSTRUMENT MONUMENT	○
MB	MONITOR BOX	□
TM	TILT METER	⊙
	PIEZOMETER WATER LEVEL AT MULTIPURPOSE POOL	○
	OPEN TUBE PIEZOMETER (7-13-89)	○
	PNEUMATIC PIEZOMETER (4-24-90)	○

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: R. G. D.

Drawn by: P. S. T.

Checked by: R. G. D. / W. B. E.

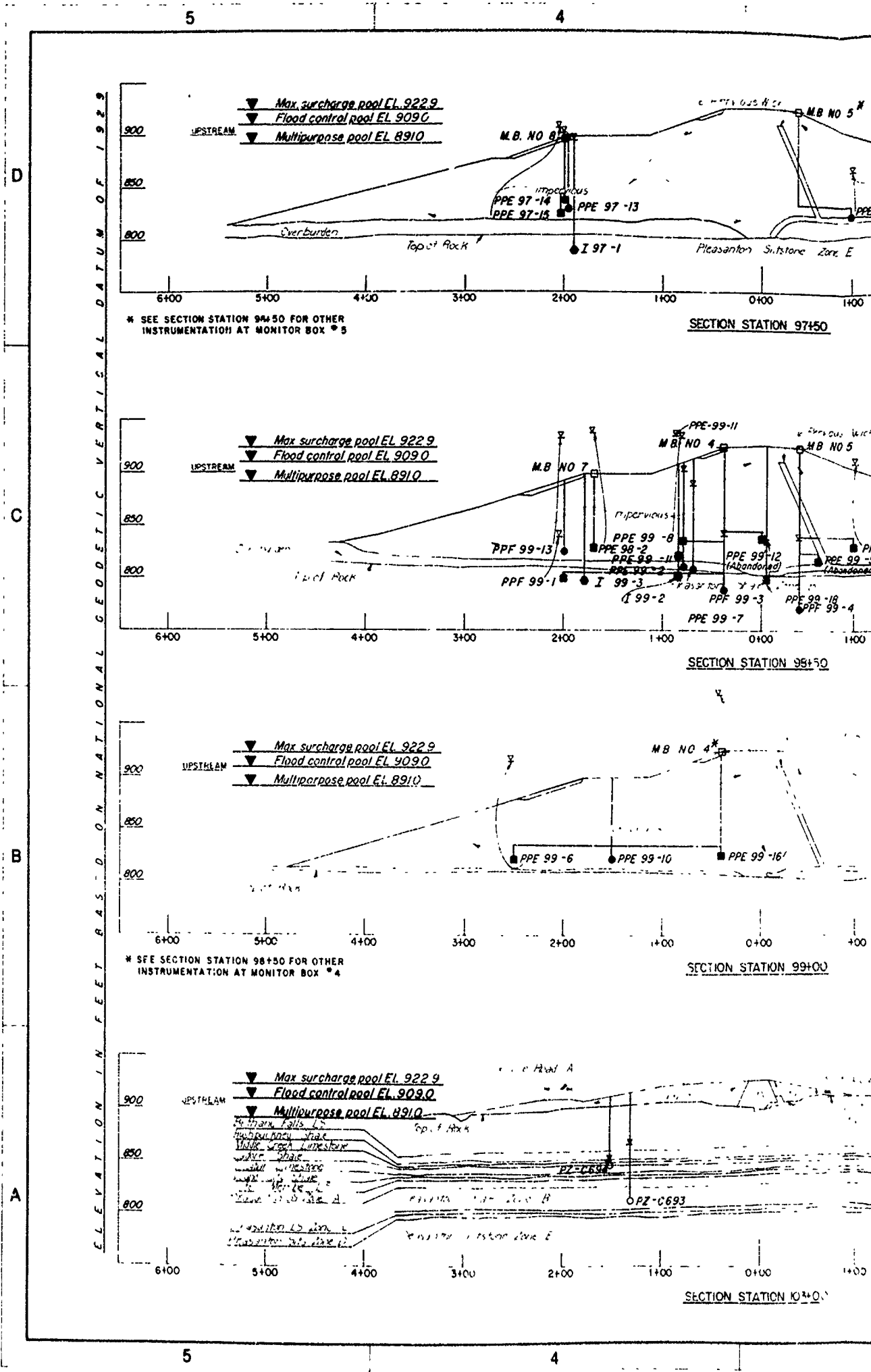
Submitted by: R. G. D.

Scale: AS SHOWN

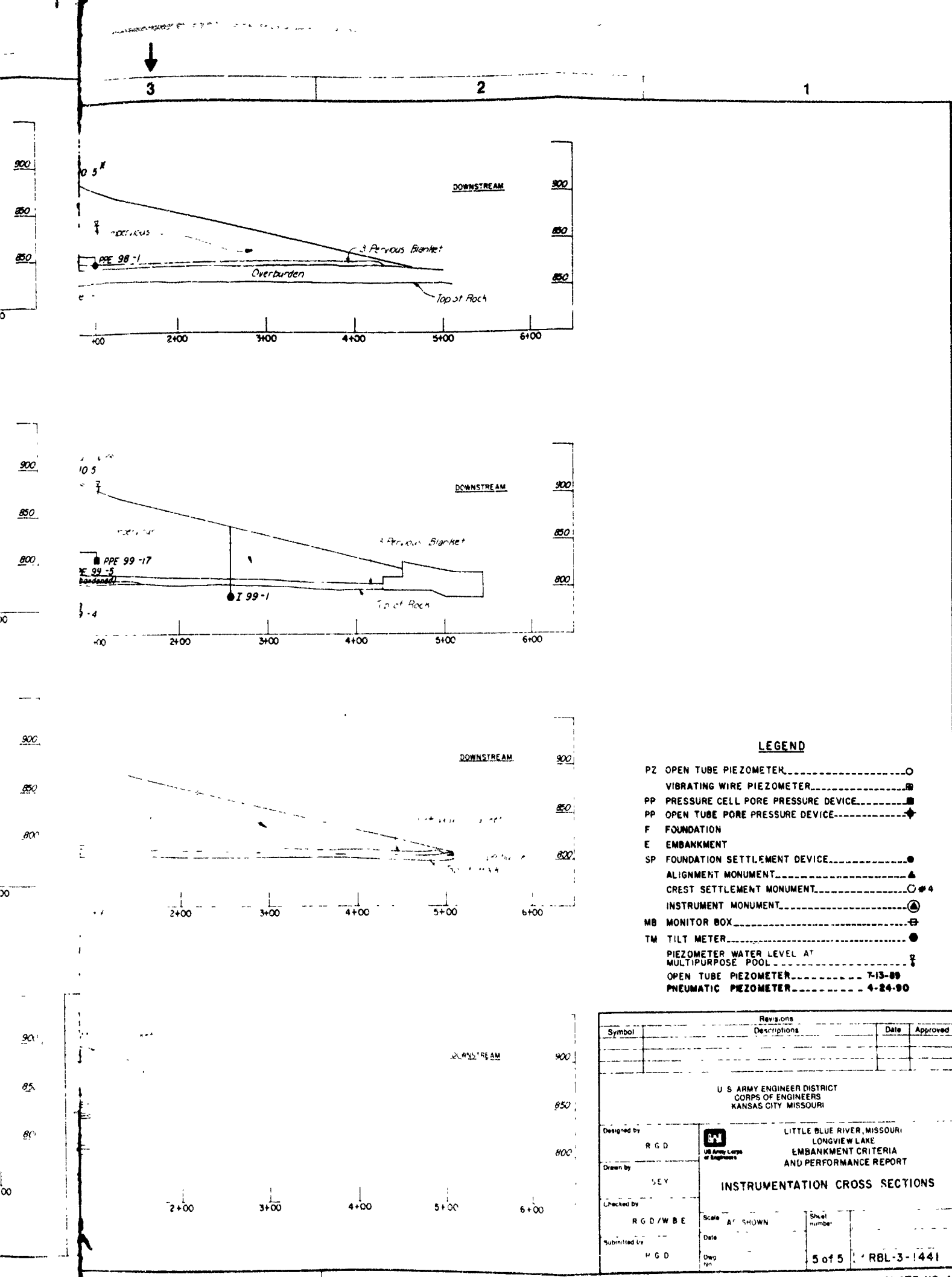
Sheet number: 4 of 5

Project number: RBL-3-1440

INSTRUMENTATION CROSS SECTIONS







# LEGEND

- PZ OPEN TUBE PIEZOMETER.....○
- VIBRATING WIRE PIEZOMETER.....■
- PP PRESSURE CELL PORE PRESSURE DEVICE.....■
- PP OPEN TUBE PORE PRESSURE DEVICE.....◆
- F FOUNDATION
- E EMBANKMENT
- SP FOUNDATION SETTLEMENT DEVICE.....●
- ALIGNMENT MONUMENT.....▲
- CREST SETTLEMENT MONUMENT.....○ #4
- INSTRUMENT MONUMENT.....⊙
- MB MONITOR BOX.....⊞
- TM TILT METER.....●
- PIEZOMETER WATER LEVEL AT MULTIPURPOSE POOL.....⊞
- OPEN TUBE PIEZOMETER.....7-13-89
- PNEUMATIC PIEZOMETER.....4-24-90

Symbol	Revisions	Date	Approved
	Descriptions		

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: R G D

Drawn by: SEV

Checked by: R G D / W B E

Submitted by: R G D

Scale: AS SHOWN

Date:

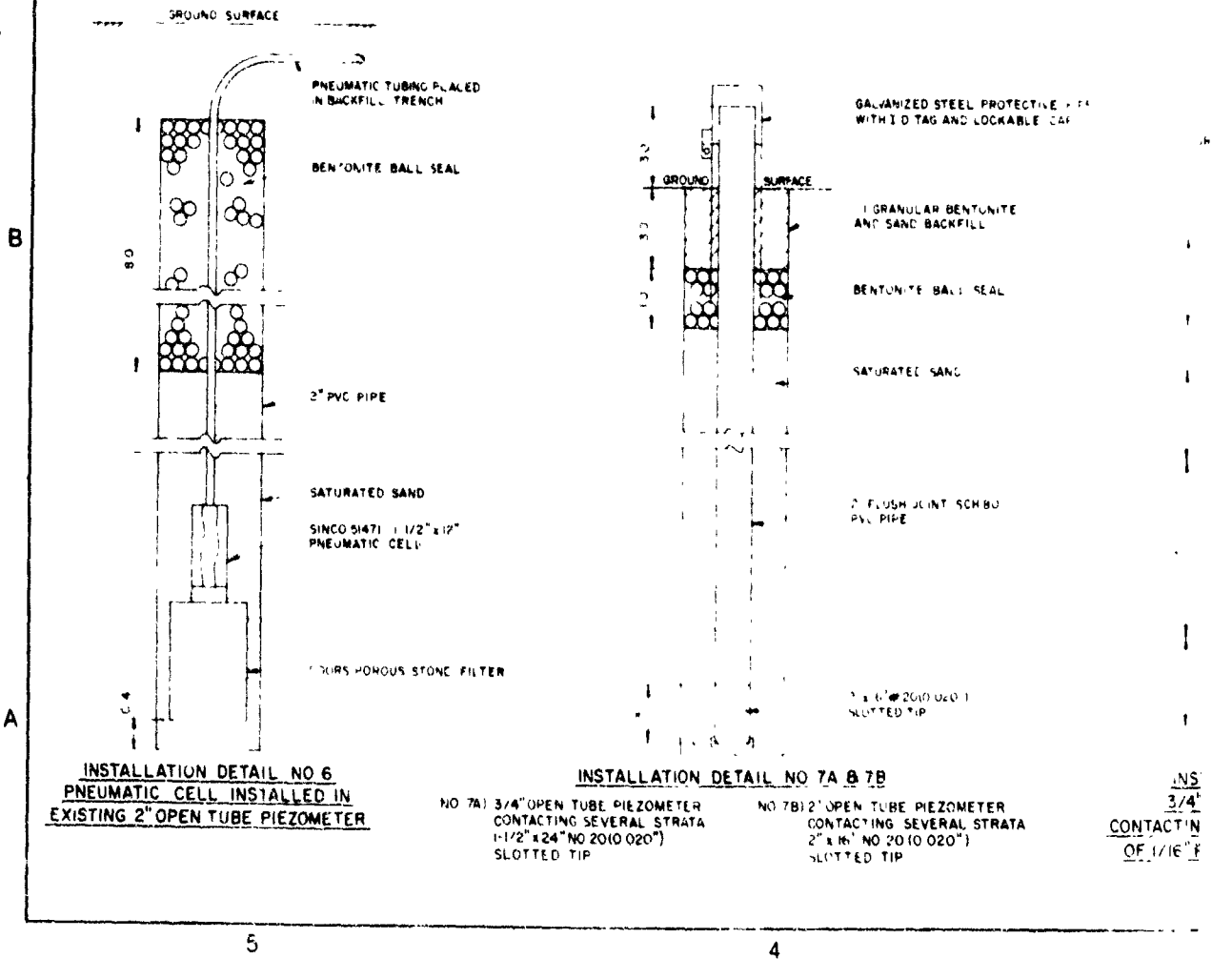
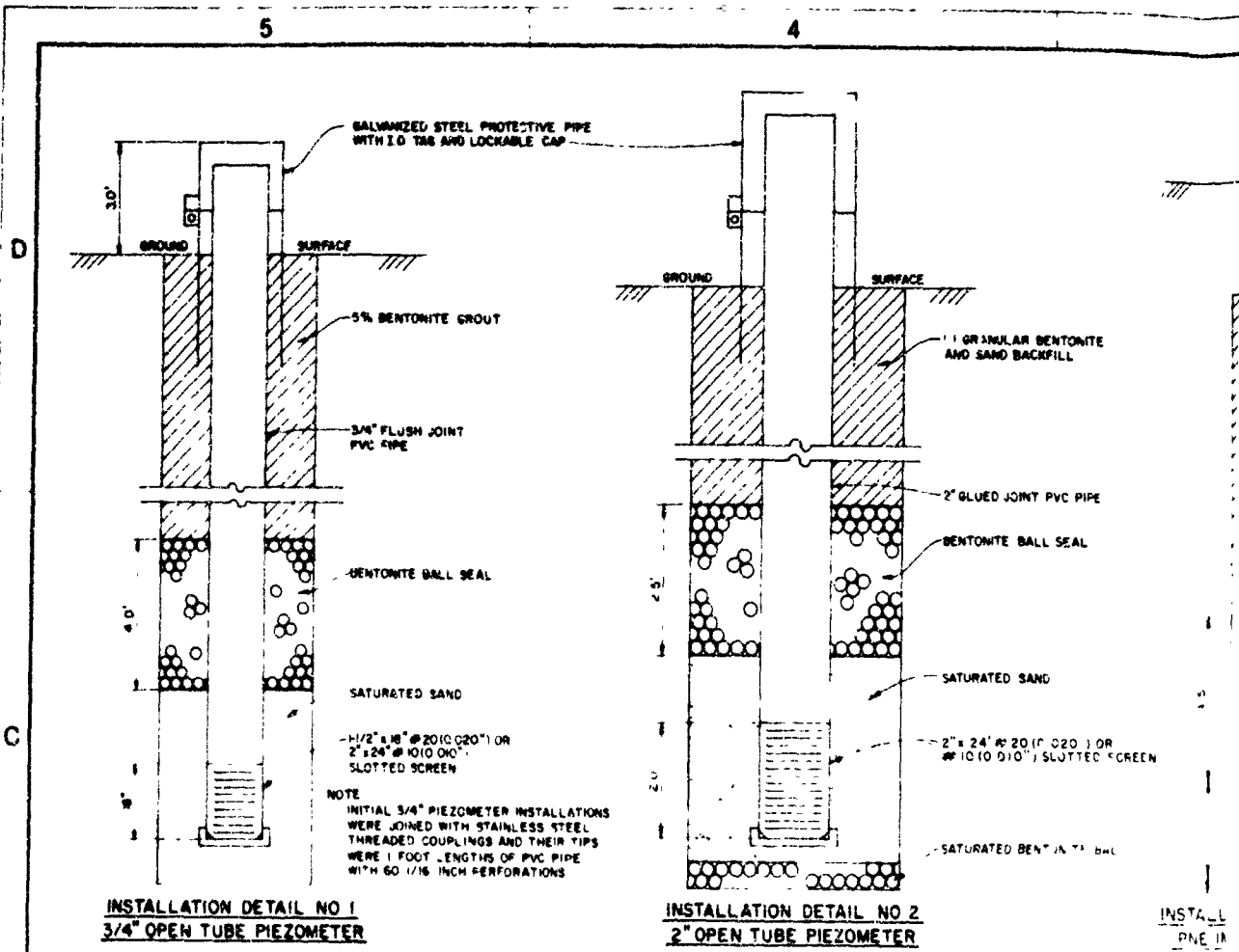
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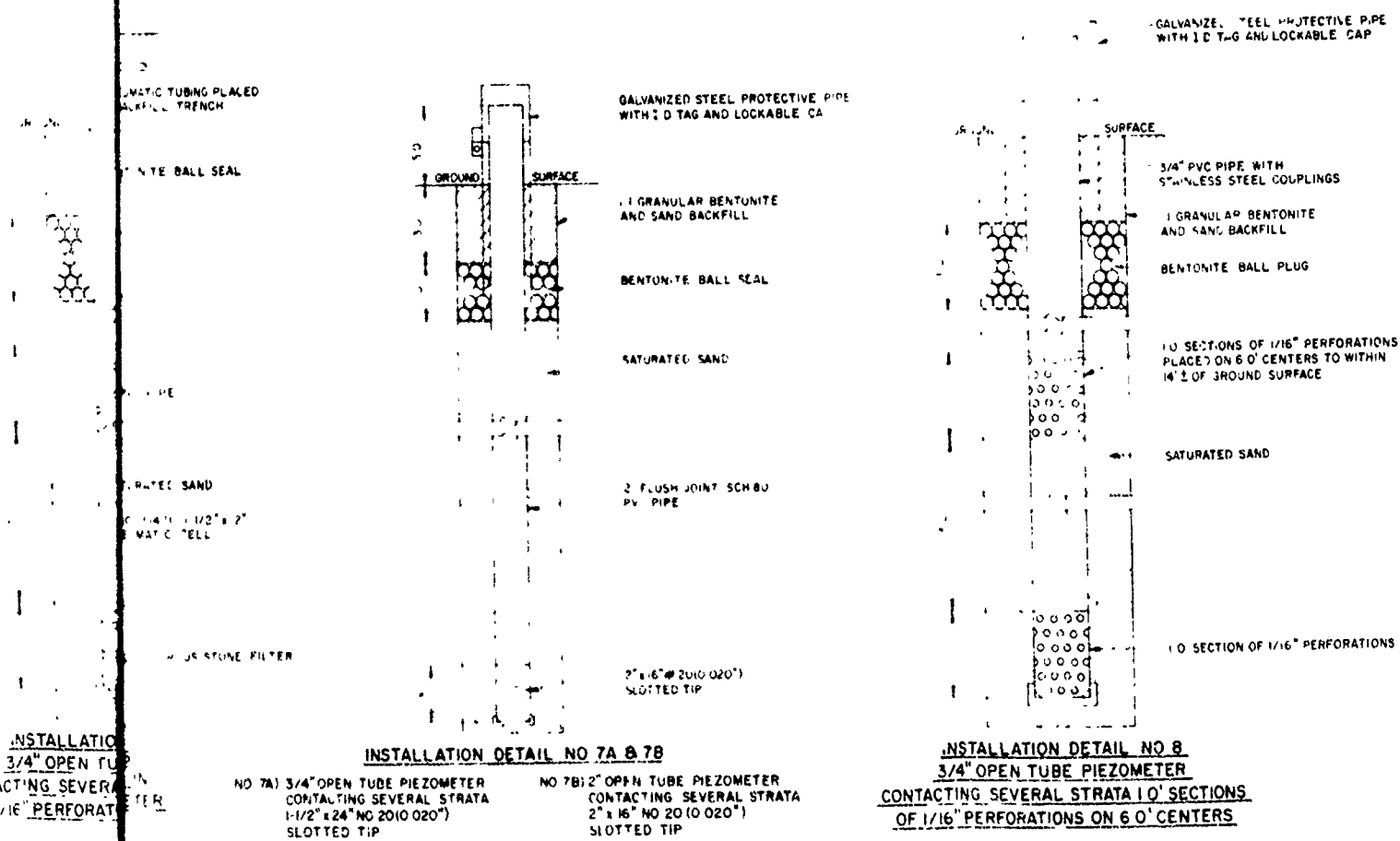
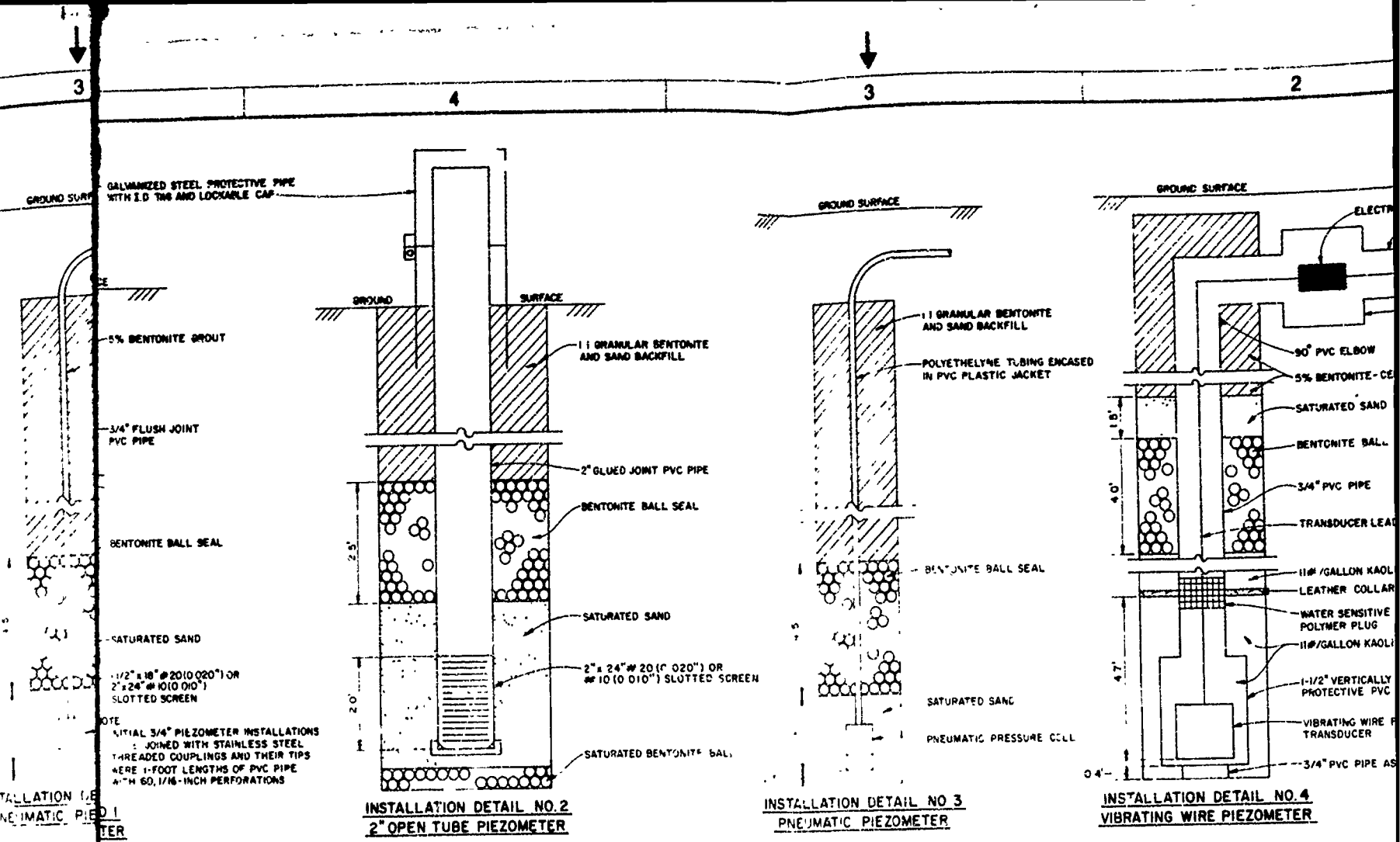
Sheet number: 5 of 5

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

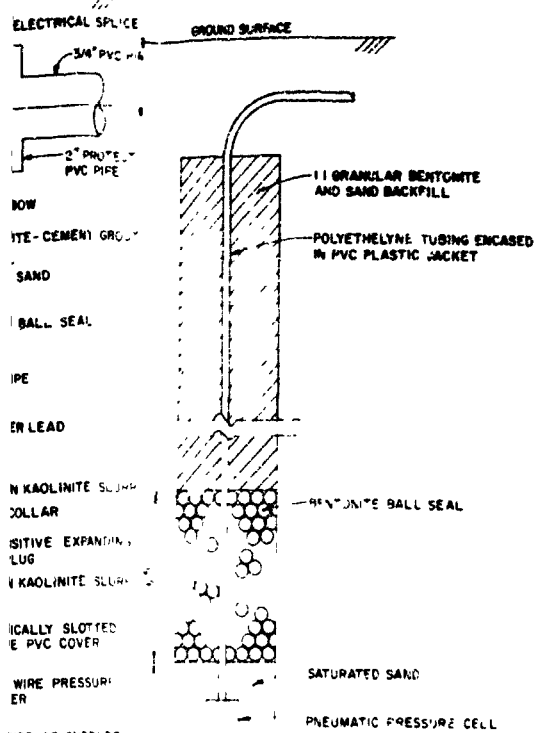
INSTRUMENTATION CROSS SECTIONS

RBL-3-1441

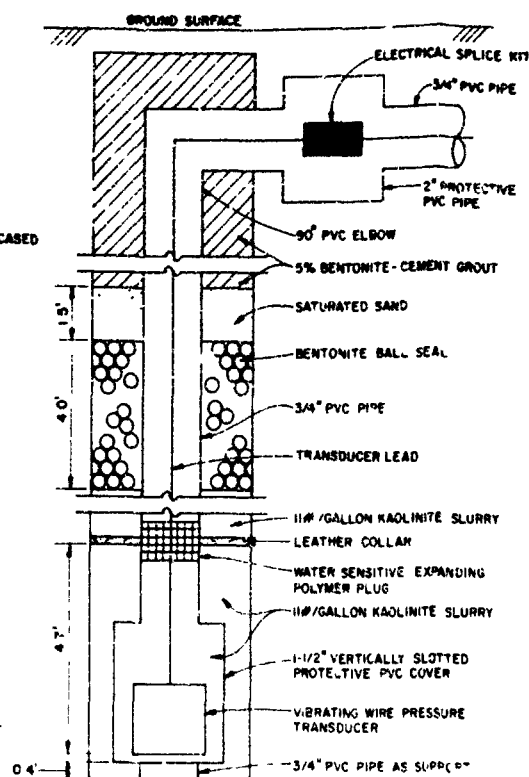




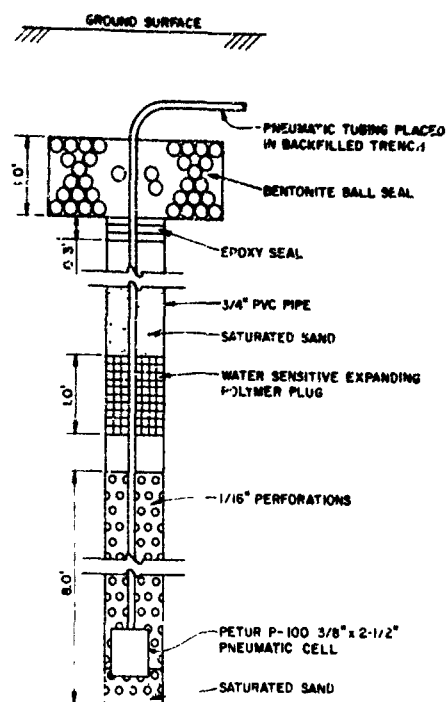




INSTALLATION DETAIL NO. 3  
PNEUMATIC PIEZOMETER

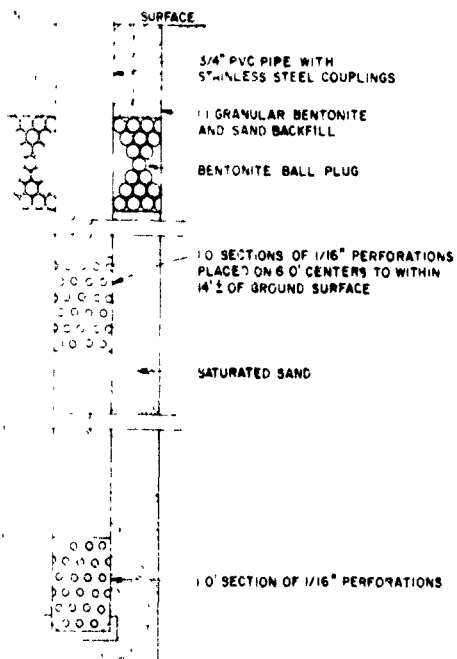


INSTALLATION DETAIL NO. 4  
VIBRATING WIRE PIEZOMETER



INSTALLATION DETAIL NO. 5  
PNEUMATIC CELL INSTALLED IN  
EXISTING 3/4" OPEN TUBE PIEZOMETER

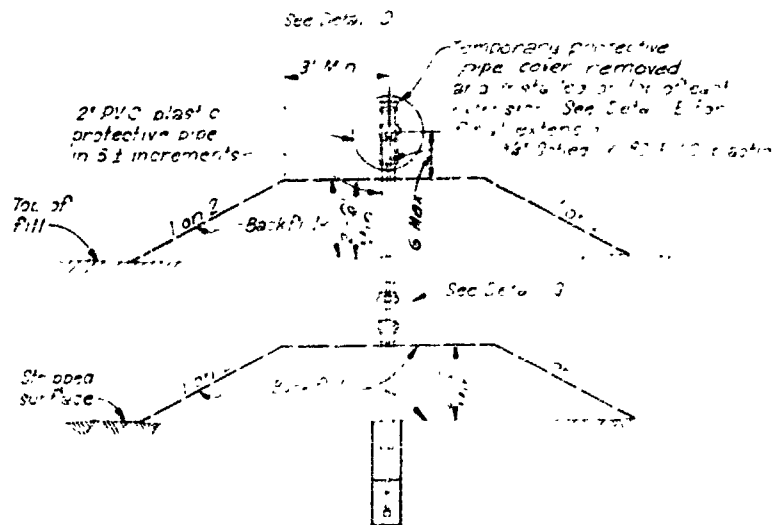
CALVANIZED STEEL PROTECTIVE PIPE  
WITH I.D. TAG AND LOCKABLE CAP



INSTALLATION DETAIL NO. 8  
3/4" OPEN TUBE PIEZOMETER  
CONTACTING SEVERAL STRATA 10' SECTIONS  
OF 1/16" PERFORATIONS ON 6' CENTERS

DETAIL NO	PIEZOMETER TYPE
1	3/4" OPEN TUBE PIEZOMETER
2	2" OPEN TUBE PIEZOMETER
3	PNEUMATIC PIEZOMETER
4	VIBRATING WIRE PIEZOMETER
5	PNEUMATIC CELL INSTALLED IN EXISTING 3/4" OPEN TUBE PIEZOMETER
6	PNEUMATIC CELL INSTALLED IN EXISTING 2" OPEN TUBE PIEZOMETER
7A	3/4" OPEN TUBE PIEZOMETER CONTACTING SEVERAL STRATA 1-1/2" x 24" #20(0.020") SLOTTED TIP
7B	2" OPEN TUBE PIEZOMETER CONTACTING SEVERAL STRATA 2" x 16" #20(0.020") SLOTTED TIP
8	3/4" OPEN TUBE PIEZOMETER CONTACTING SEVERAL STRATA 10' SECTIONS OF 1/16" PERFORATIONS ON 6' CENTERS

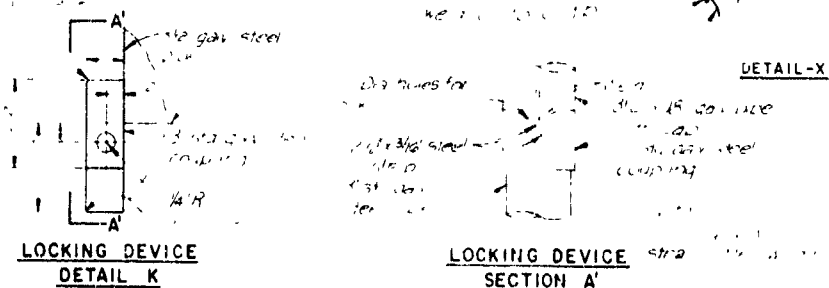
Symbol	Revisions	Date	Approved
	Descriptions		
<p>U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p> <p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</p> <p>INSTRUMENTATION INSTALLATION DETAILS</p> <p>1 of 2 RBL-3-1442</p>			



**OPEN TUBE PIEZOMETER  
INSTALLATION AND EXTENSION  
DETAIL-A**

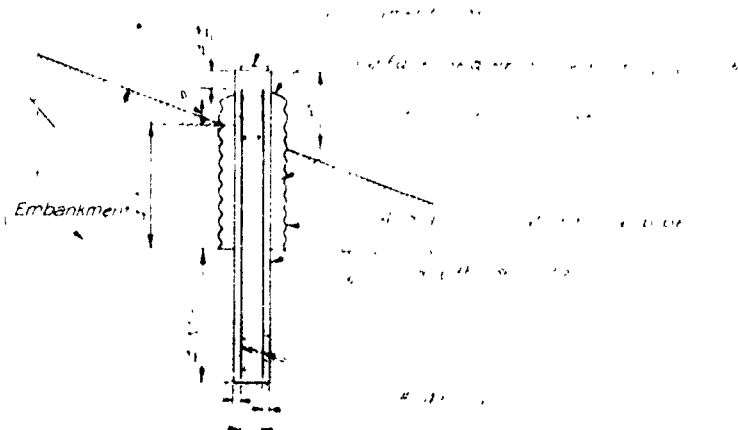


**FINAL POSITION OF PIEZOMETERS  
AND SETTLEMENT DEVICES  
DETAIL-E**

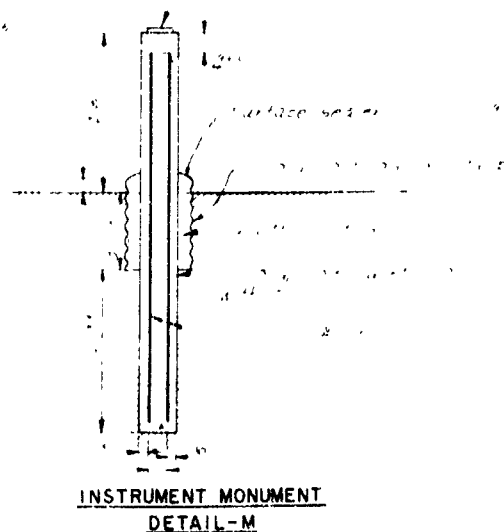


**LOCKING DEVICE  
DETAIL K**

**LOCKING DEVICE  
SECTION A'**



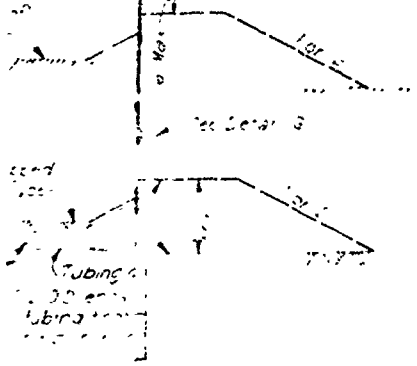
**ALIGNMENT MONUMENTS  
DETAIL-L**



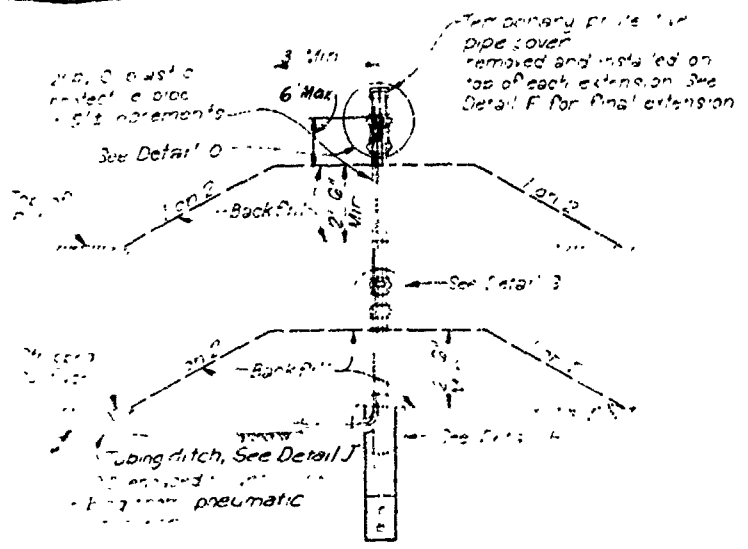
**INSTRUMENT MONUMENT  
DETAIL-M**

**MONITOR BO  
DETAIL-F**

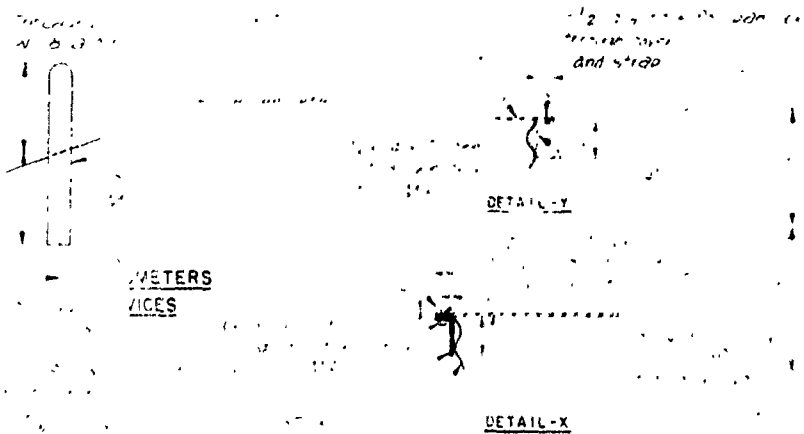
Temporary protective  
pipe cover removed  
and installed on top of each  
extension. See Detail E for  
extension. See Detail F for  
final extension.



PIEZOMETER  
WITH TUBING EXTENSION  
DETAIL-A



OPEN TUBE PIEZOMETER  
WITH TUBING FROM PNEUMATIC PIEZOMETER  
DETAIL-C



METERS  
ICES

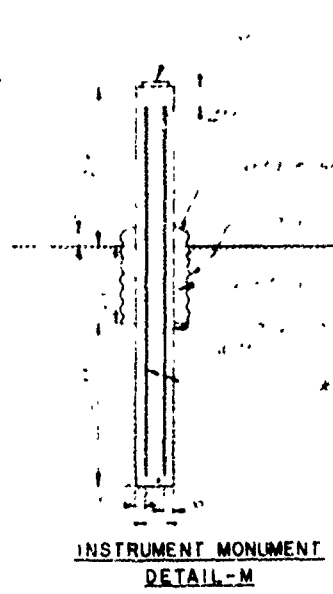
BOX  
-F

MONITOR BOX  
DETAIL-E

PROTECTIVE  
PIPE COUPLING  
DETAIL-G

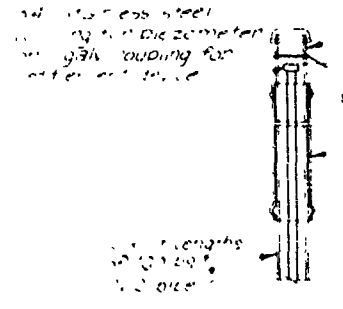
TEE CONNecT  
DETAIL-I

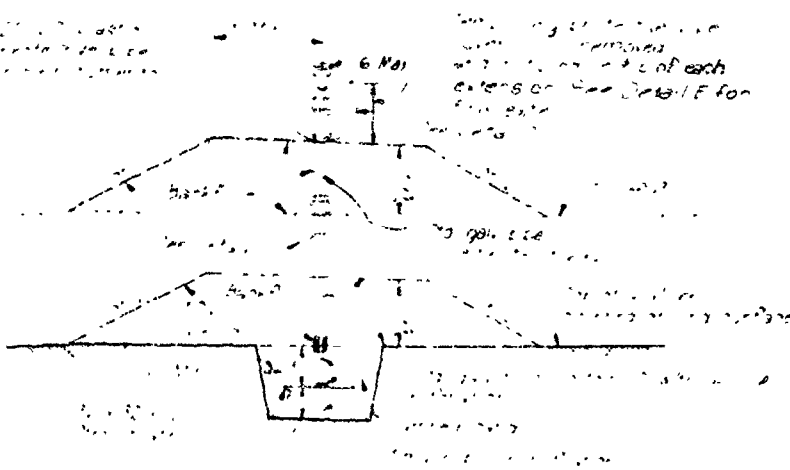
LOCKING DEVICE  
SECTION A



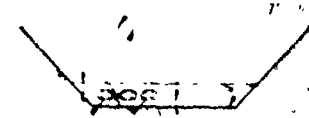
CREST SETTLEMENT MONUMENT  
DETAIL-N

TEMPORARY  
PROTECTIVE PIPE  
DETAIL-O

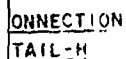




SETTLEMENT DEVICE INSTALLATION AND EXTENSION  
DETAIL-D



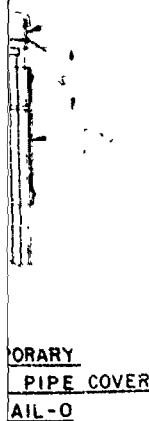
TUBING DITCH  
DETAIL - J



MAIL BOX  
11L-E

PROTECTIVE  
PIPE COUPLING  
DETAIL-G

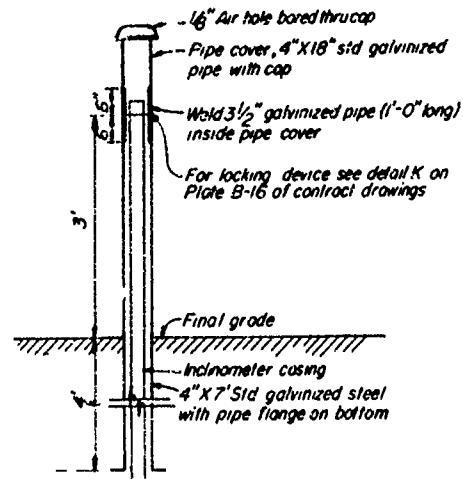
TEE CONNECTION  
DETAIL-H



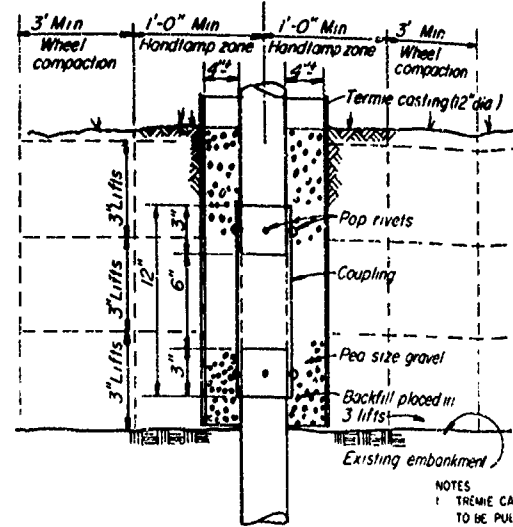
CREST SETTLEMENT MONUMENT  
DETAIL-N

TEMPORARY  
PROTECTIVE PIPE COVER  
DETAIL-0

[illegible]



**INCLINOMETER PROTECTIVE PIPE DETAIL**



**INCLINOMETER EXTENSION**

- NOTES
1. TREMIE CASING TO BE PULLED
  2. MAKE SURE GROOVES IN TREMIE PE
  3. TREMIE PE CASING, IN
  4. POP RIVET TO STAND

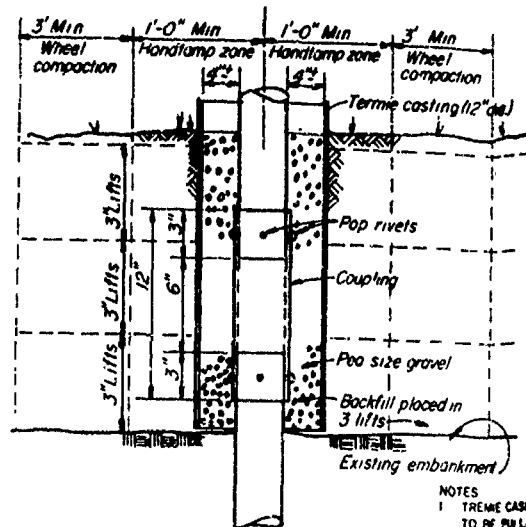
or hole bored thru cap  
cover, 4" X 18" std galvanized  
with cap

1/2" galvanized pipe (1'-0" long)  
pipe cover

locking device see detail K on  
B-16 of contract drawings

grade  
meter casing  
Std galvanized steel  
pe flange on bottom

# PIPE DETAIL



## INCLINOMETER EXTENSION

### NOTES

1. TREME CASING (12" DIA PLASTIC PIPE (SCHEDULE 40))  
TO BE PULLED AS EXTENSION PROGRESSES
2. MAKE SURE THAT THE FOUR INTERNAL LONGITUDINAL  
GROOVES IN UPPER AND LOWER CASING MATCH UP
3. TREME PEA SIZE GRAVEL AROUND INCLINOMETER  
CASING, IN AN OUTER PROTECTIVE PIPE
4. POP RIVET UPPER AND LOWER INCLINOMETER CASING  
TO STANDARD COUPLING IN FOUR PLACES

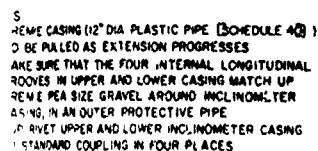
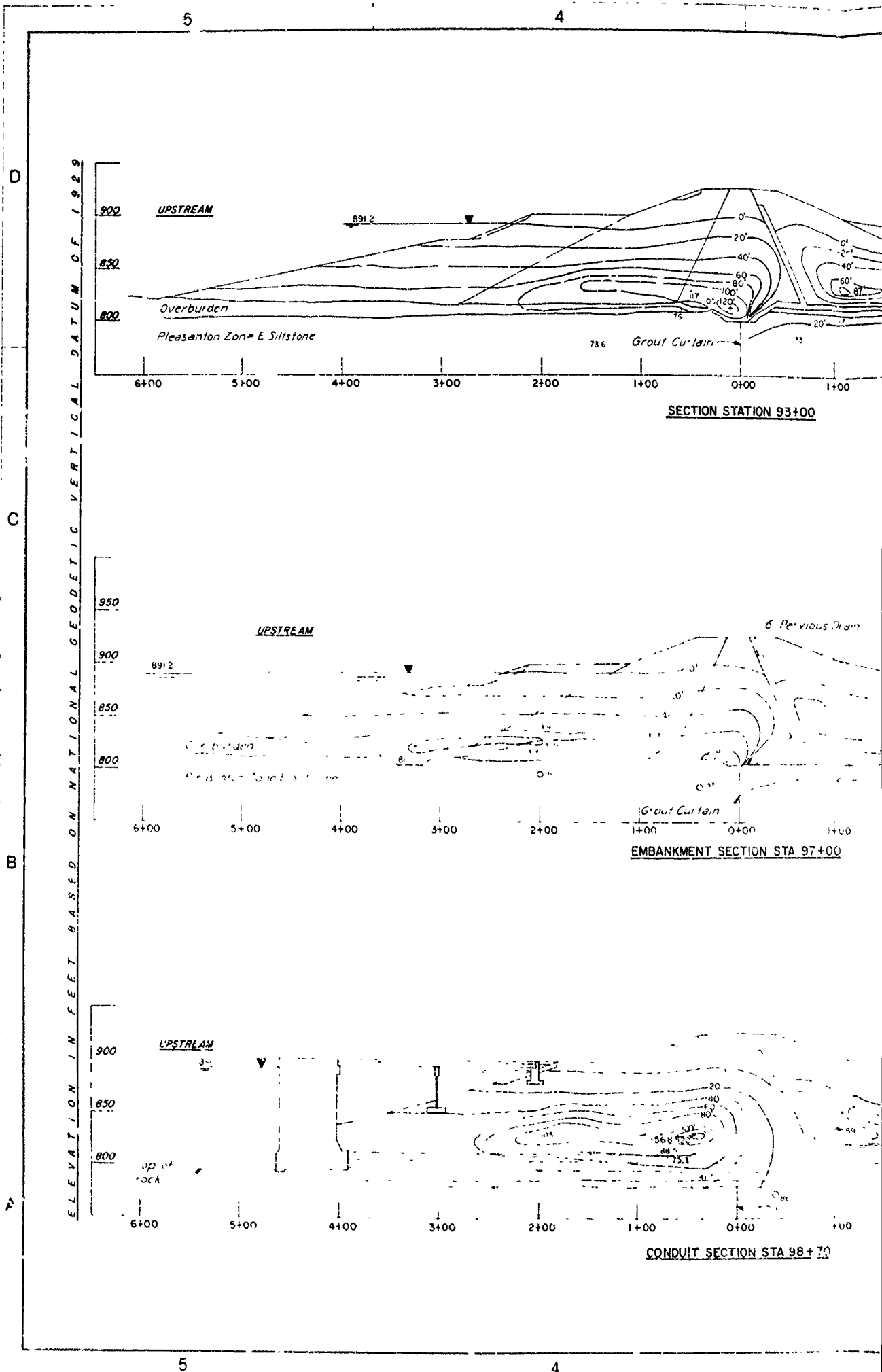
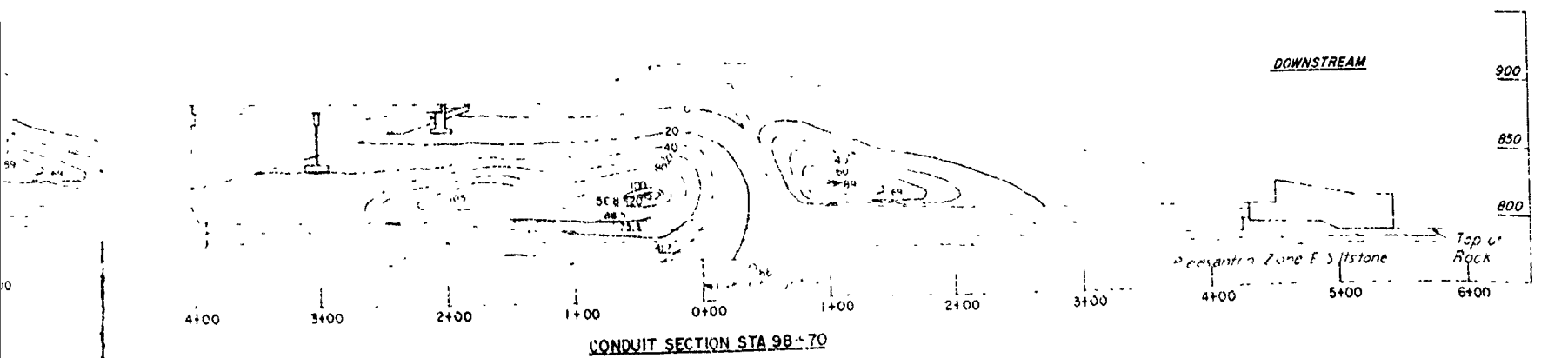
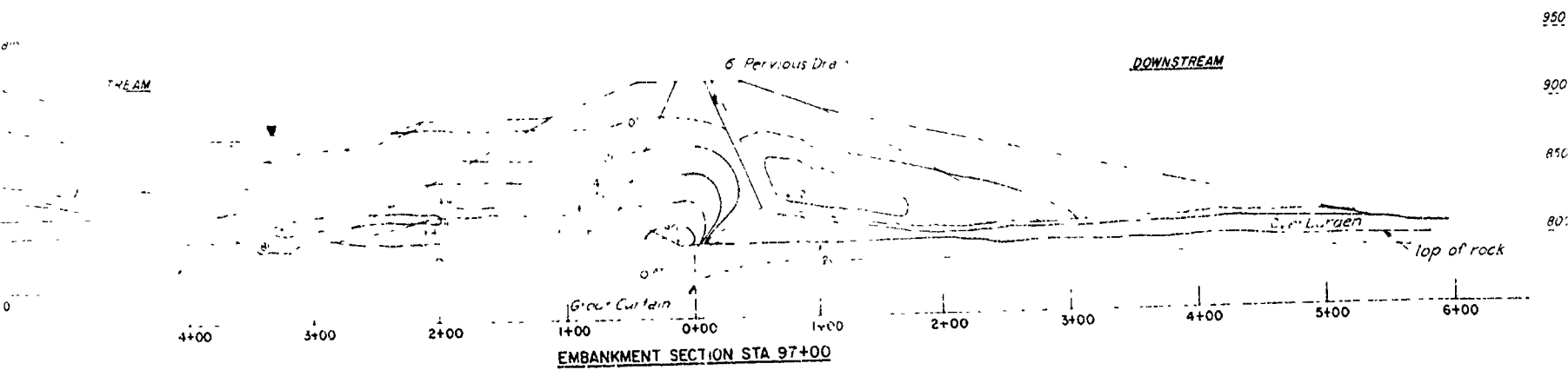
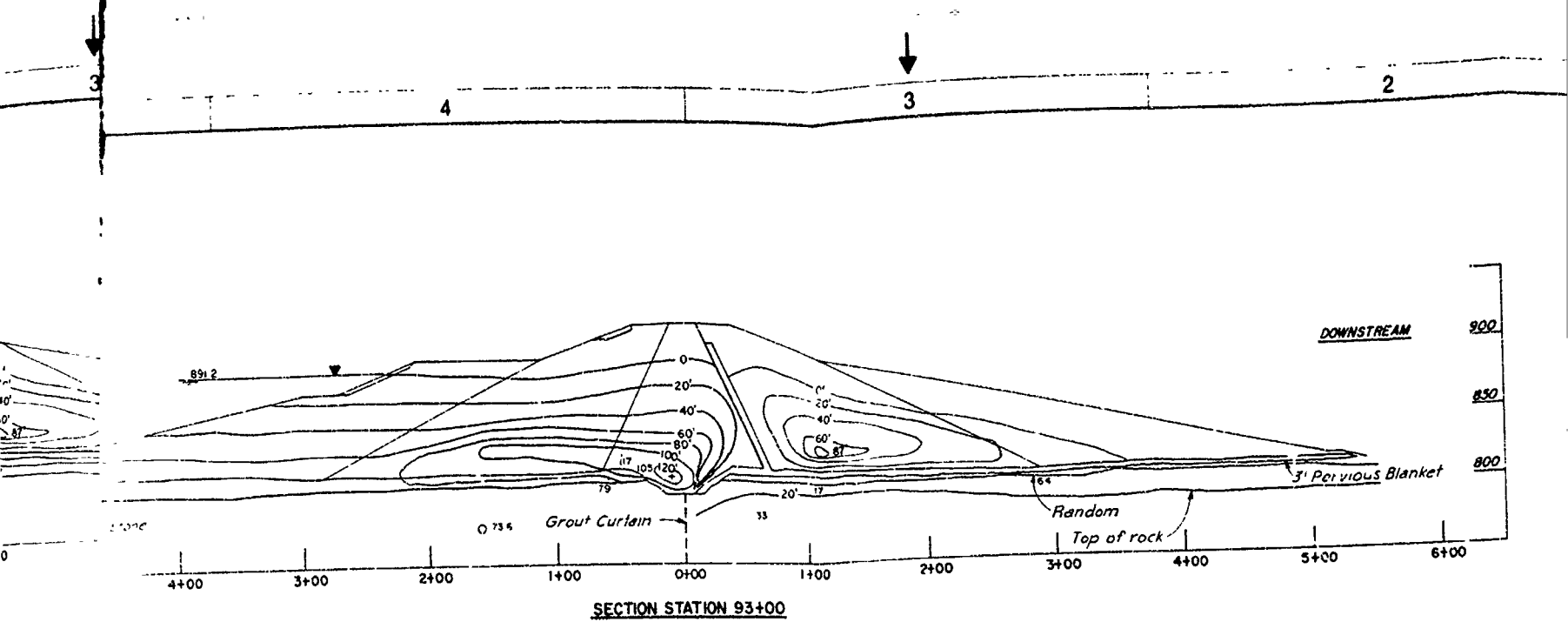
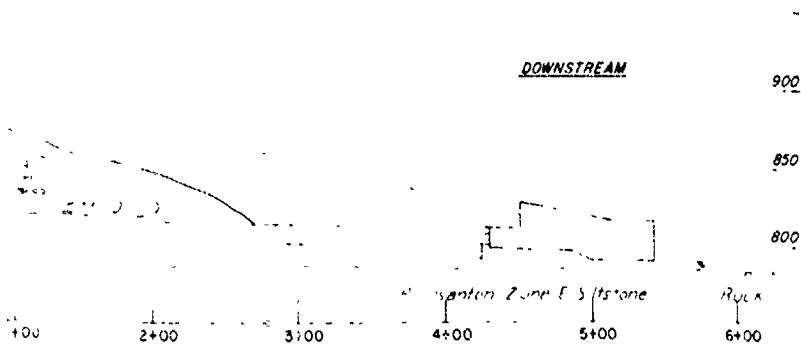
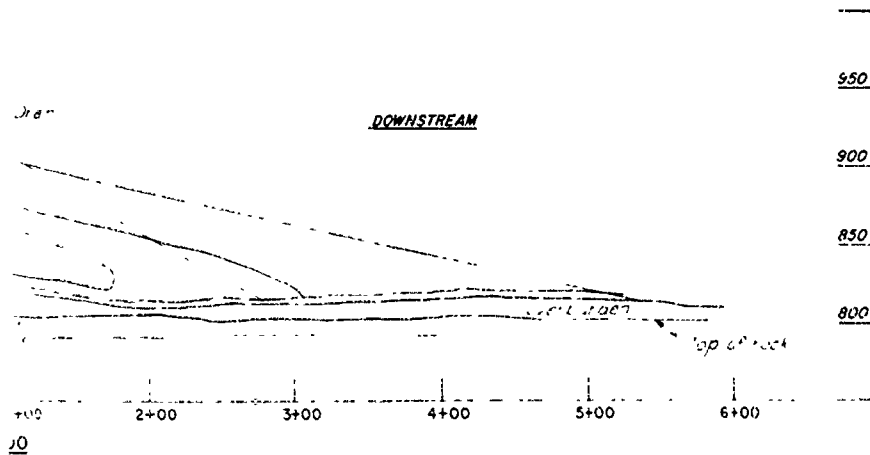
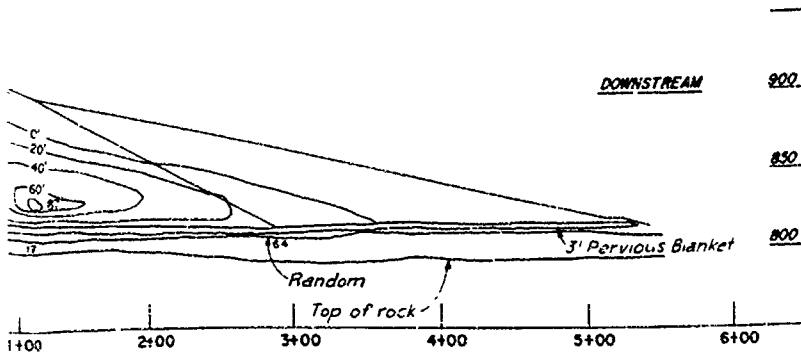


PLATE NO 43A

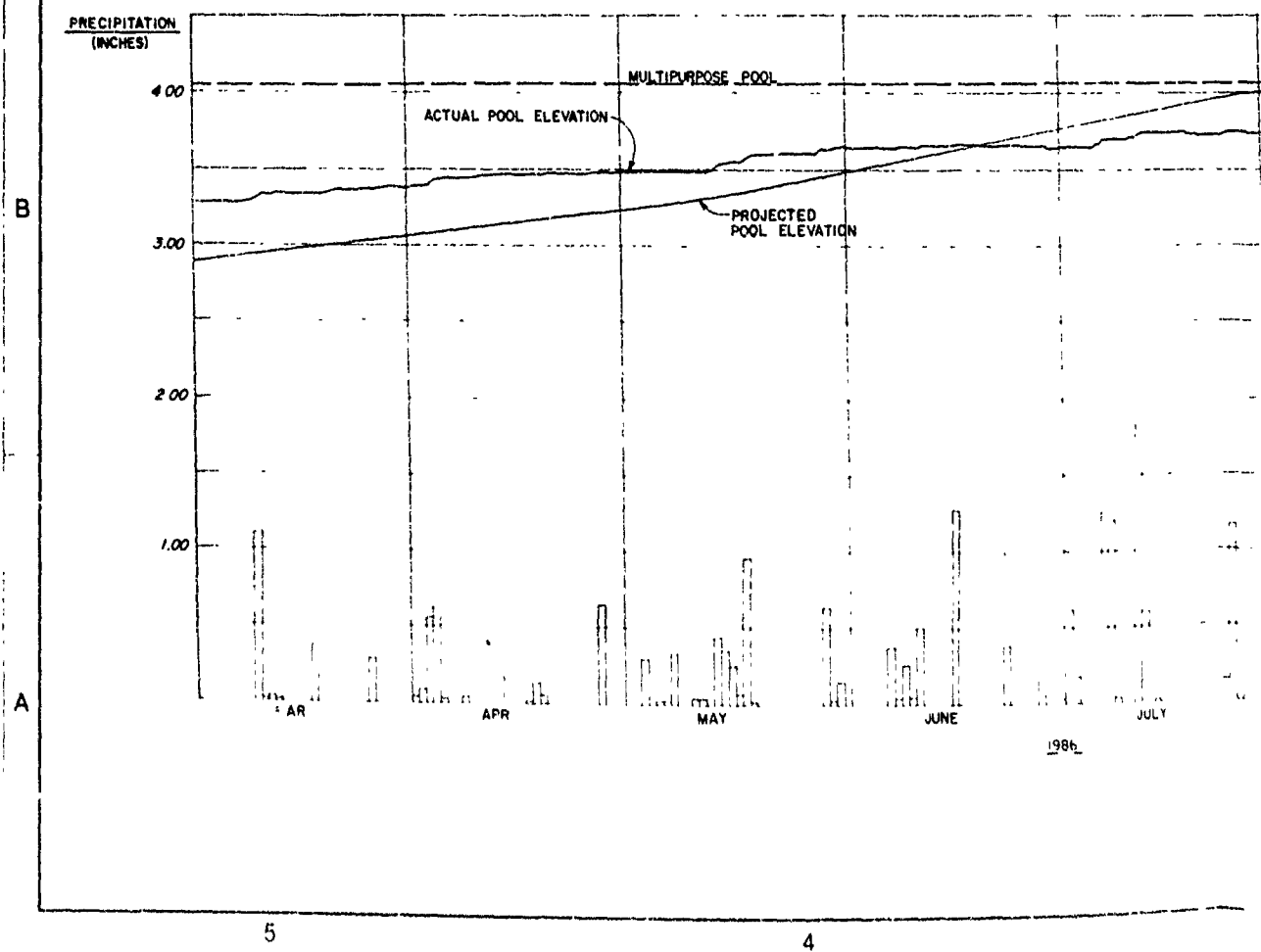
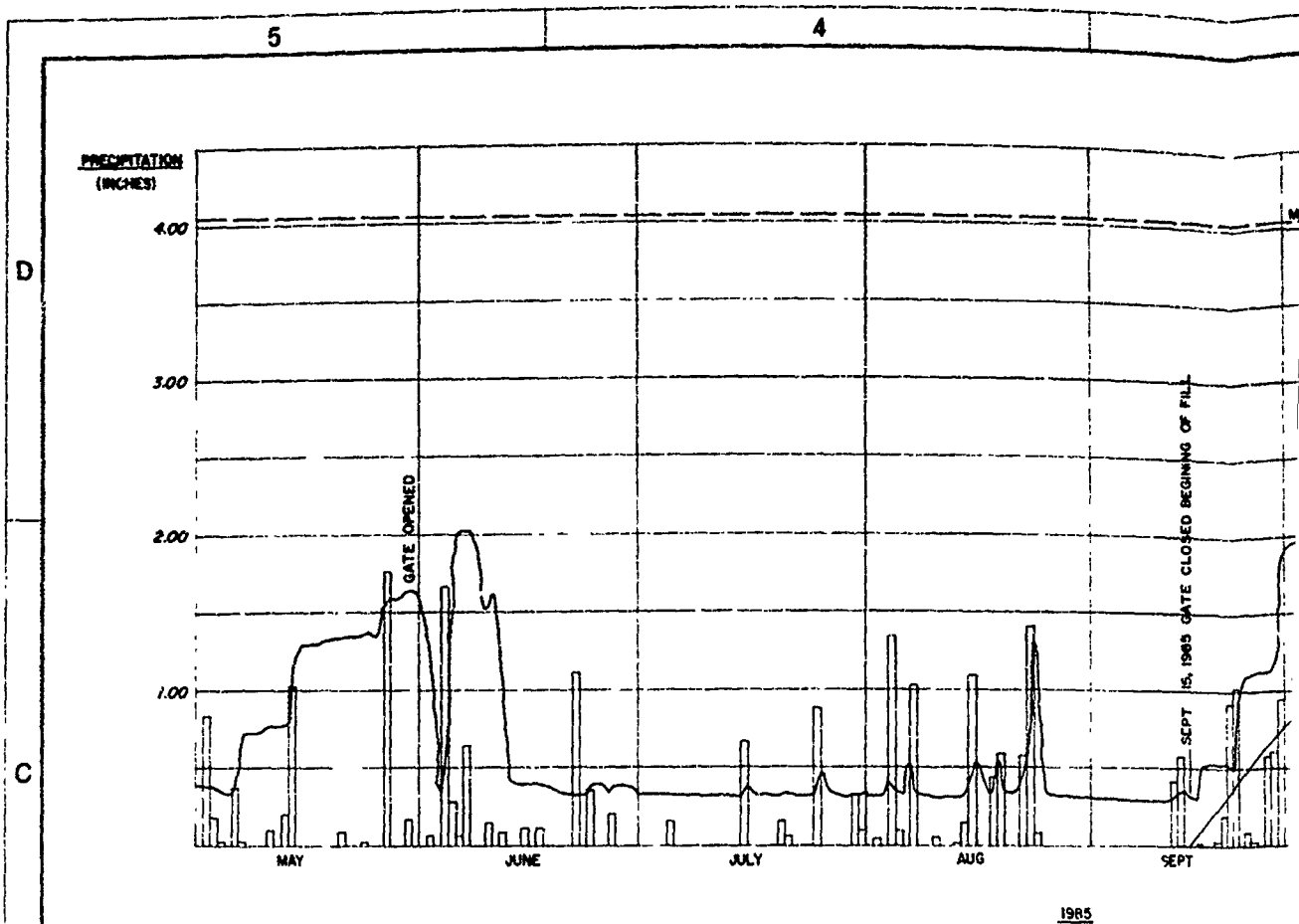


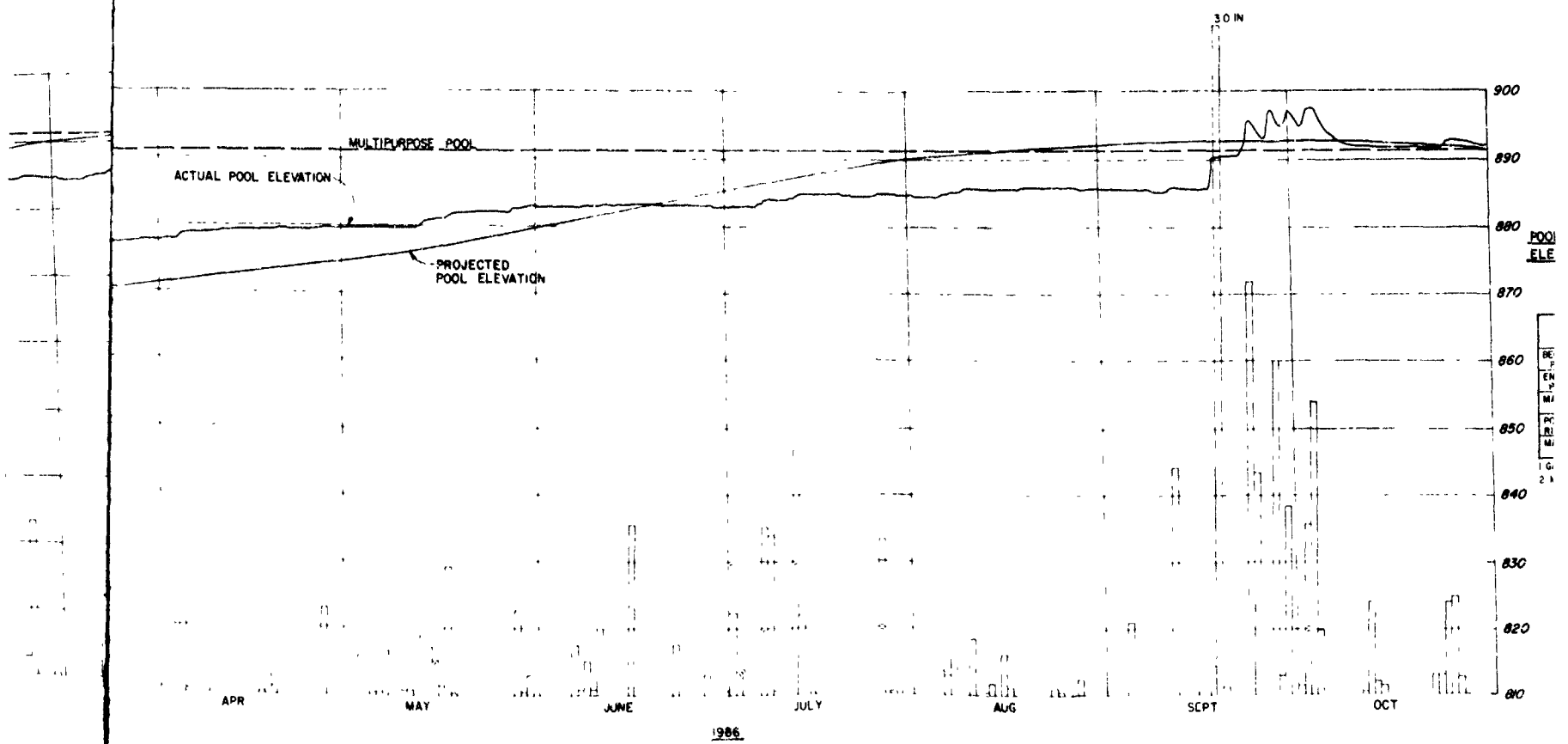
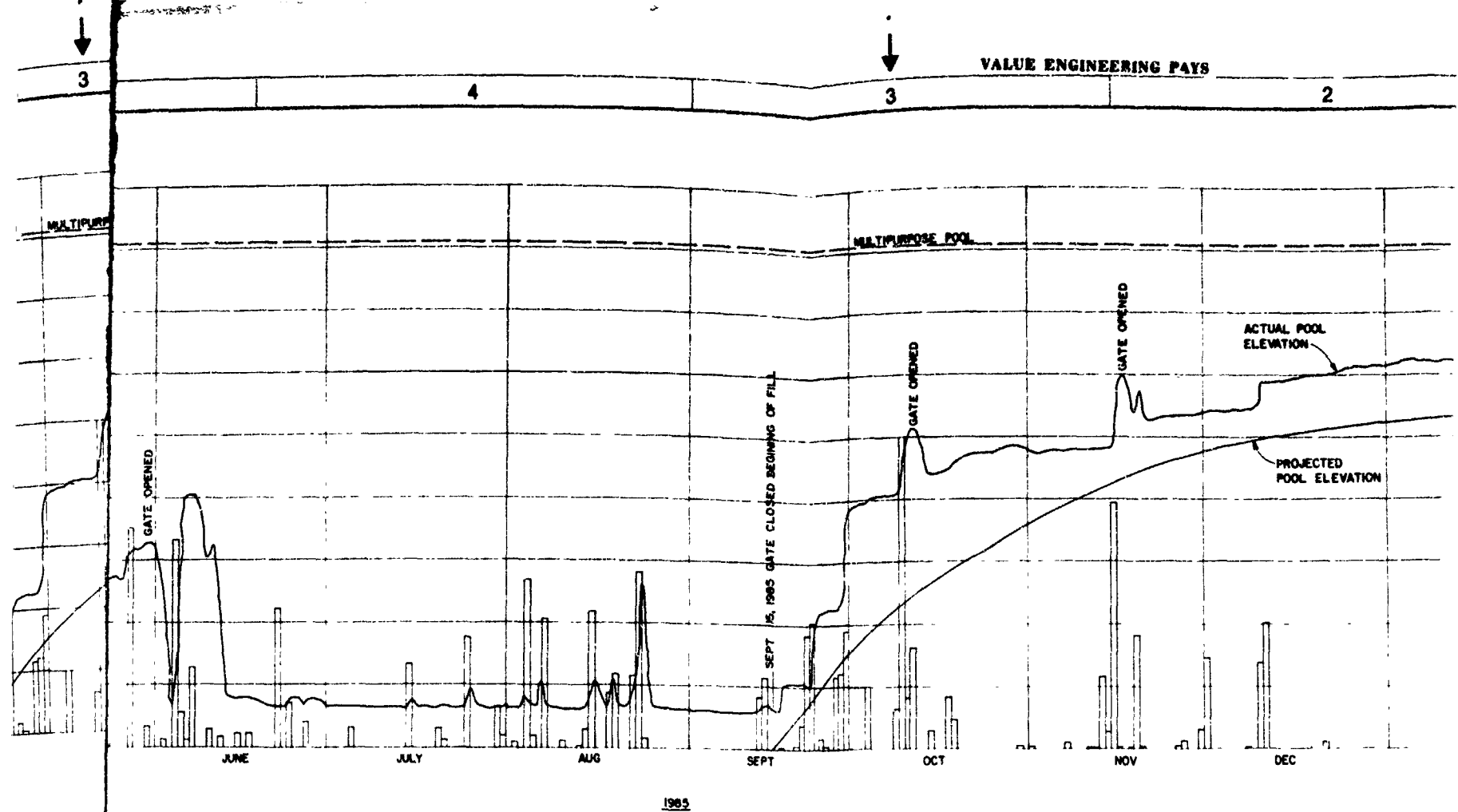






Symbol	Reviser's Descriptions	Date	Approved
<p>U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p>			
Designation:	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE</p>		
Location:	<p>EMBANKMENT CRITERIA AND PERFORMANCE REPORT</p>		
Prepared by:	<p>PORE PRESSURE CONTOURS (JANUARY 1987) POOL EL. 891.2</p>		
Reviewed by:	AS SH WN	Check initials	
Drawn by:	Date		
Scale:	Draw No.		
			RBL-3-1444

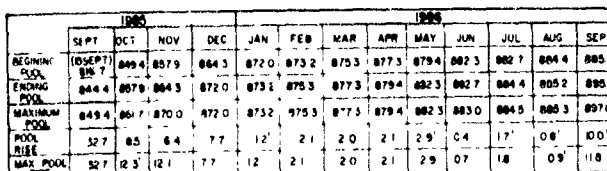




## 3

2

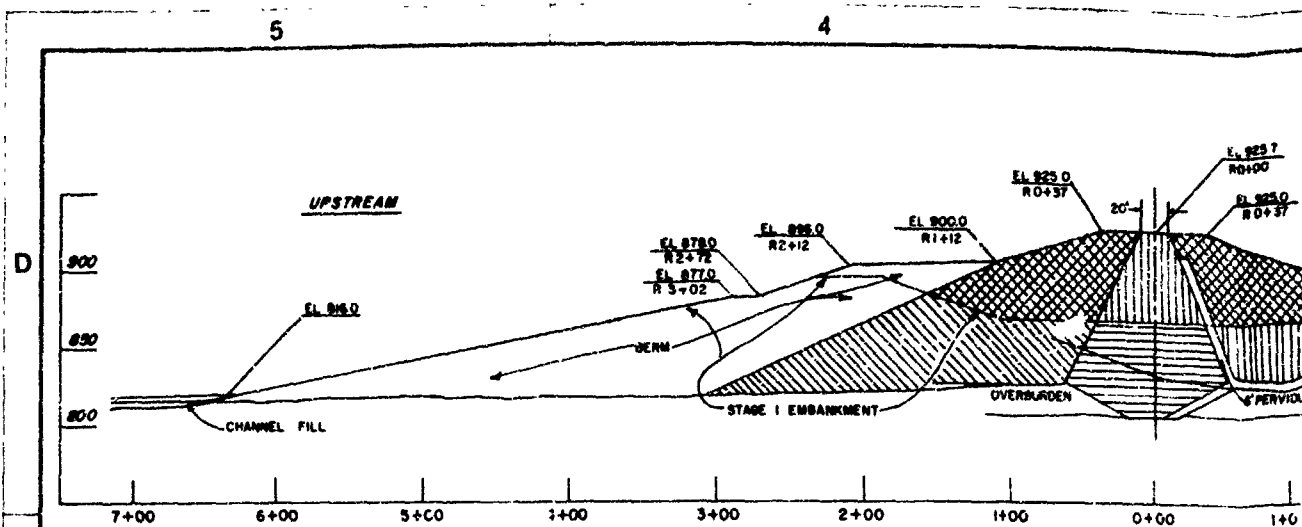
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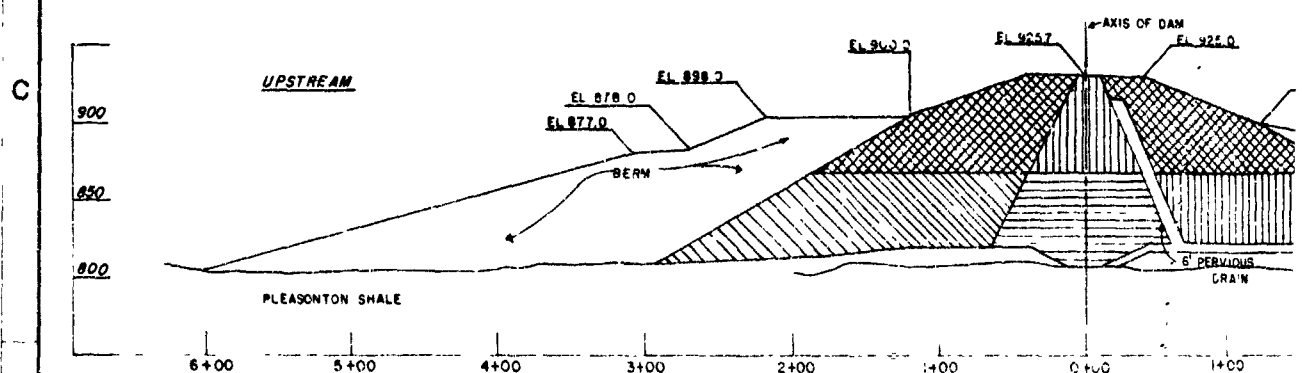
1 GATES CLOSED 15 SEPT 1983  
2 LAKE THERMOCLINE POOL REACHED 22 SEPT 1983

[illegible]

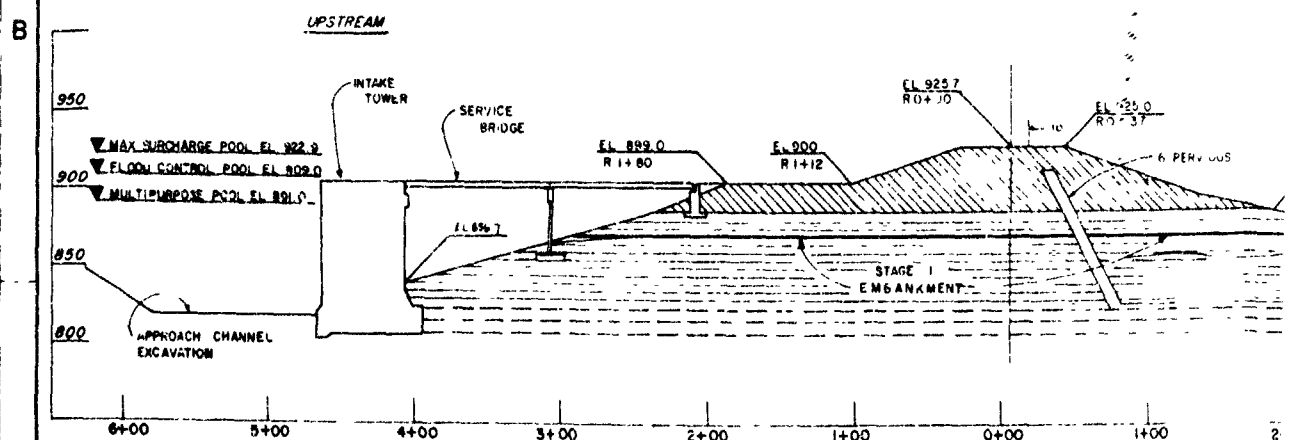
FLATE NO 45



VALLEY SECTION - STA 90+00

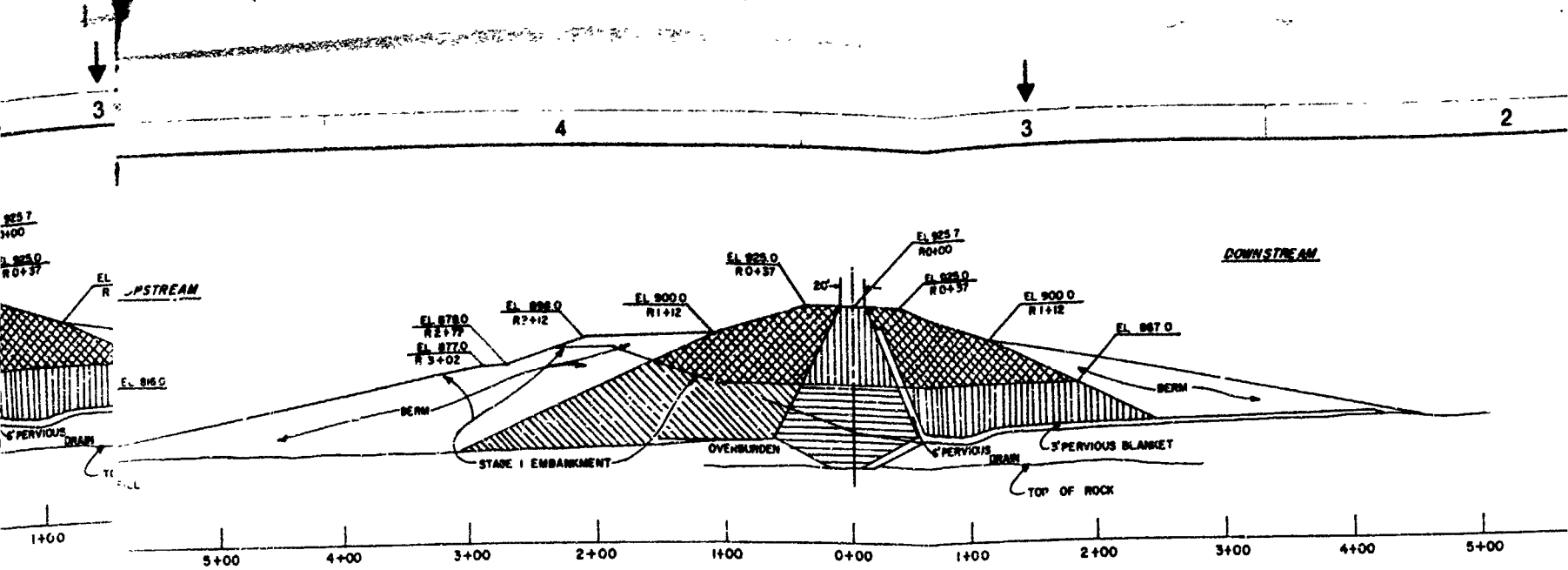


CLOSURE SECTION - STA 93+00

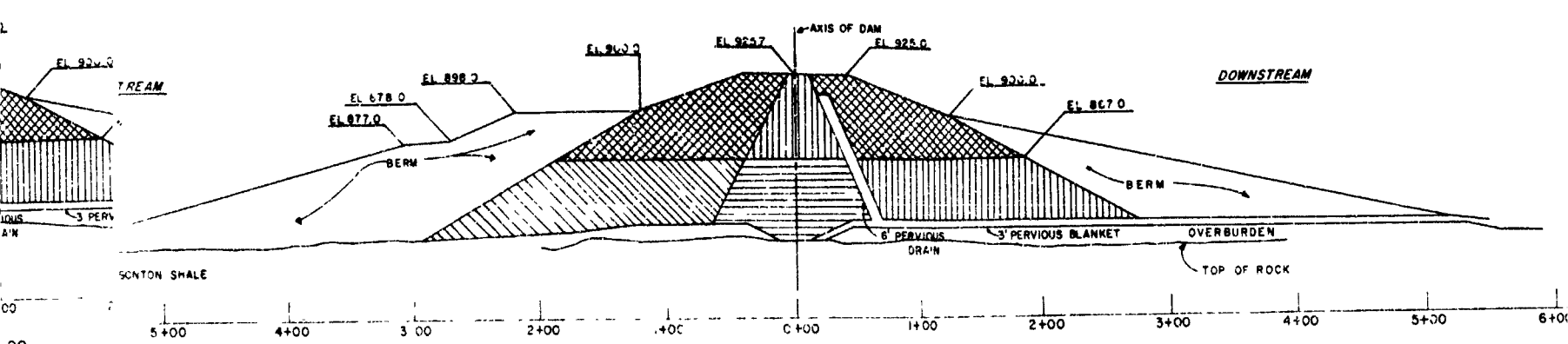


CONDUIT SECTION - STA 98+70

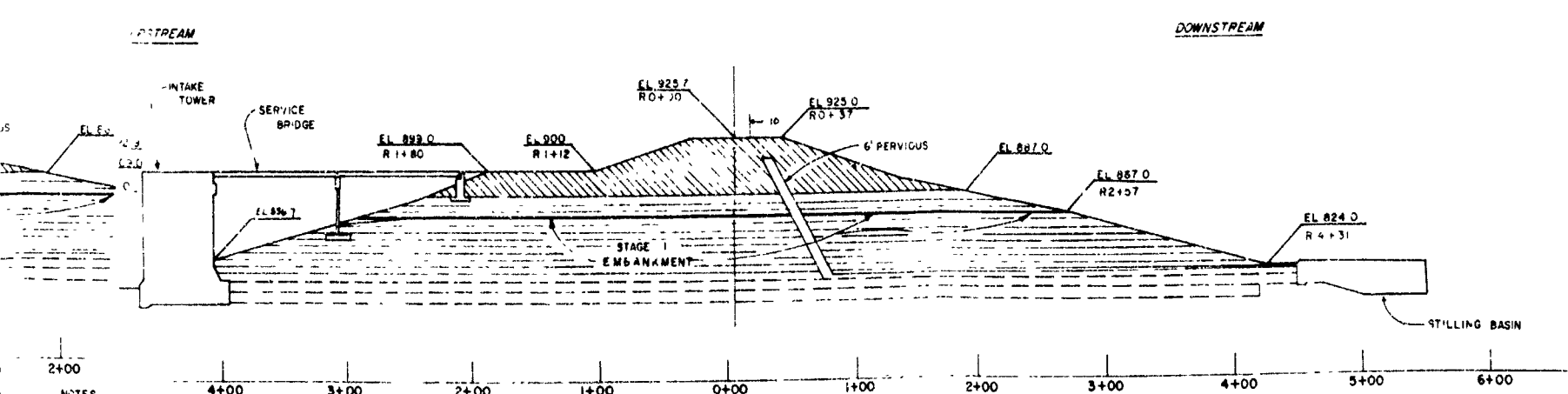




VALLEY SECTION - STA 90+00

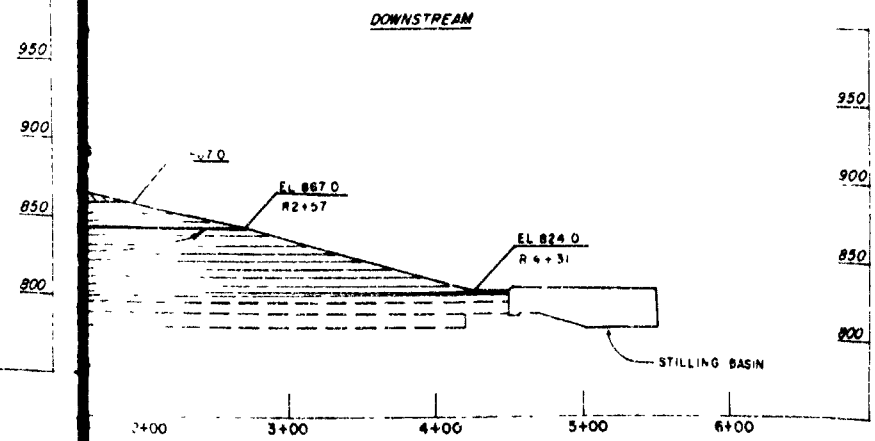
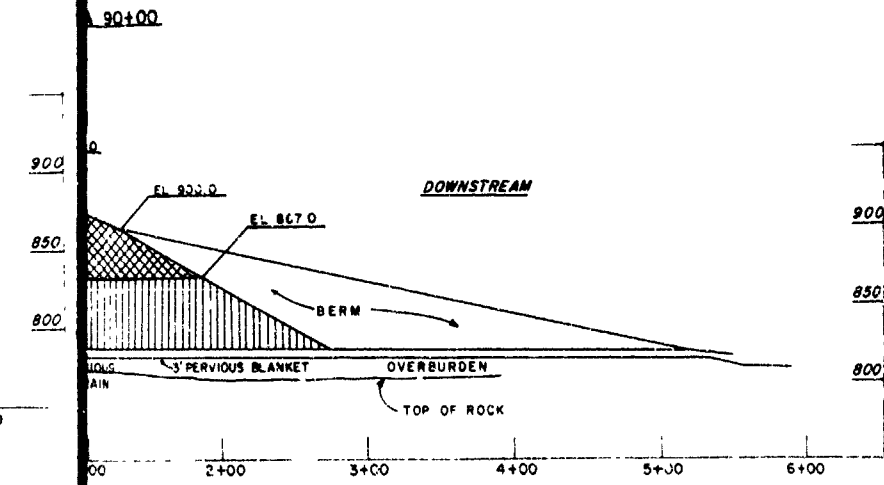
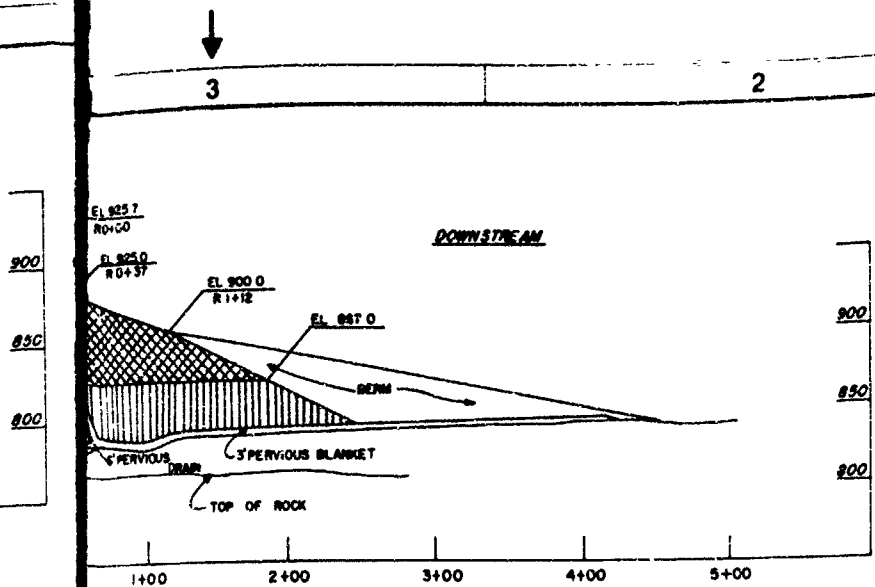


CLOSURE SECTION - STA 93+00



CONDUIT SECTION - STA 98+70

- NOTES
- 1 EMBANKMENT ZONING APPLIES BETWEEN STA 97+00 AND STA 101+00
  - 2 CROSS-SECTION APPLIES BETWEEN STA 98+00 AND STA 100+00



- NOTES
1. EMBANKMENT ZONING APPLIES BETWEEN STA 97+00 AND STA 101+00
  2. CROSS-SECTION APPLIES BETWEEN STA 98+00 AND STA 100+00

SYMBOL	ALLOWABLE LIQUID LIMIT	ZONE
	55	<ul style="list-style-type: none"> <li>IMPERVIOUS, BELOW 887.0, INCLUDING CUTOFF TRENCH</li> <li>IMPERVIOUS BETWEEN STA 97+00 AND STA 101+00 - BELOW EL 887.0</li> <li>RIVERBANK EXCAVATION BACKFILL</li> <li>IMPERVIOUS, FINISH STAGE I - NO CUTOFF TRENCH</li> </ul>
	60	<ul style="list-style-type: none"> <li>IMPERVIOUS, CUTOFF TRENCH ABOVE EL 867.0</li> <li>IMPERVIOUS, INSPECTION TRENCH</li> <li>IMPERVIOUS, BETWEEN STA 97+00 AND STA 101+00 - ABOVE EL 887.0</li> <li>RANDOM, BELOW EL 867.0 UPSTREAM OF DAM AXIS</li> </ul>
	65	<ul style="list-style-type: none"> <li>IMPERVIOUS, ABOVE EL 867.0, EXCLUDING CUTOFF TRENCH, INSPECTION TRENCH, &amp; BETWEEN STA 97+00 &amp; STA 101+00</li> <li>IMPERVIOUS, CLAY BLANKET - RT ABUTMENT</li> <li>RANDOM, BELOW EL 887.0 DOWNSTREAM OF DAM AXIS</li> </ul>
	70	<ul style="list-style-type: none"> <li>RANDOM, ABOVE EL 867.0</li> </ul>

Revisions		Date	Approved
Symbol	Descriptions		

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

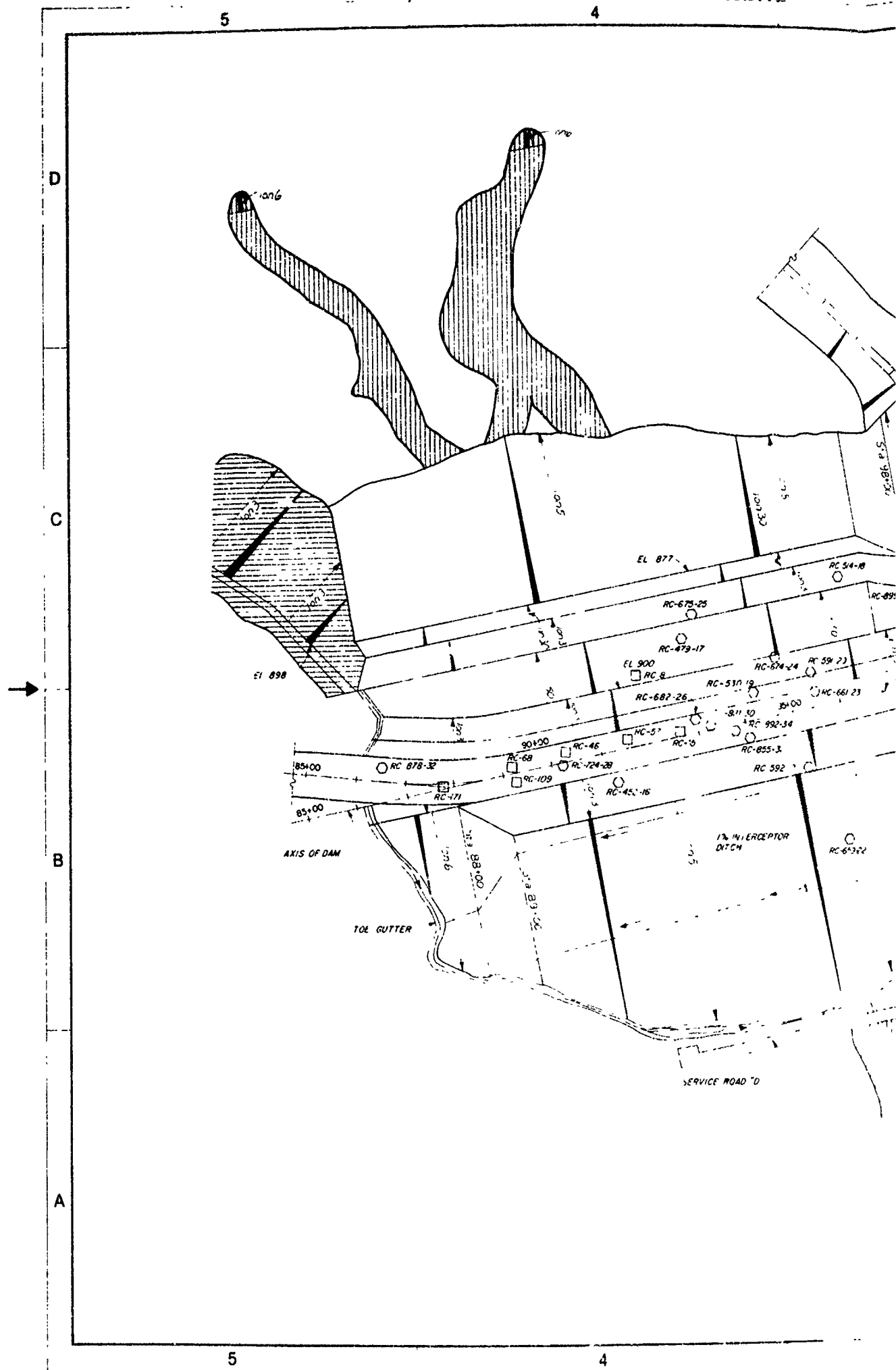
Designated by: WBE  
Drawn by: MAW  
Checked by: WBE/RGD  
In Charge: FCW

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

LIQUID LIMIT ZONE  
CROSS SECTIONS

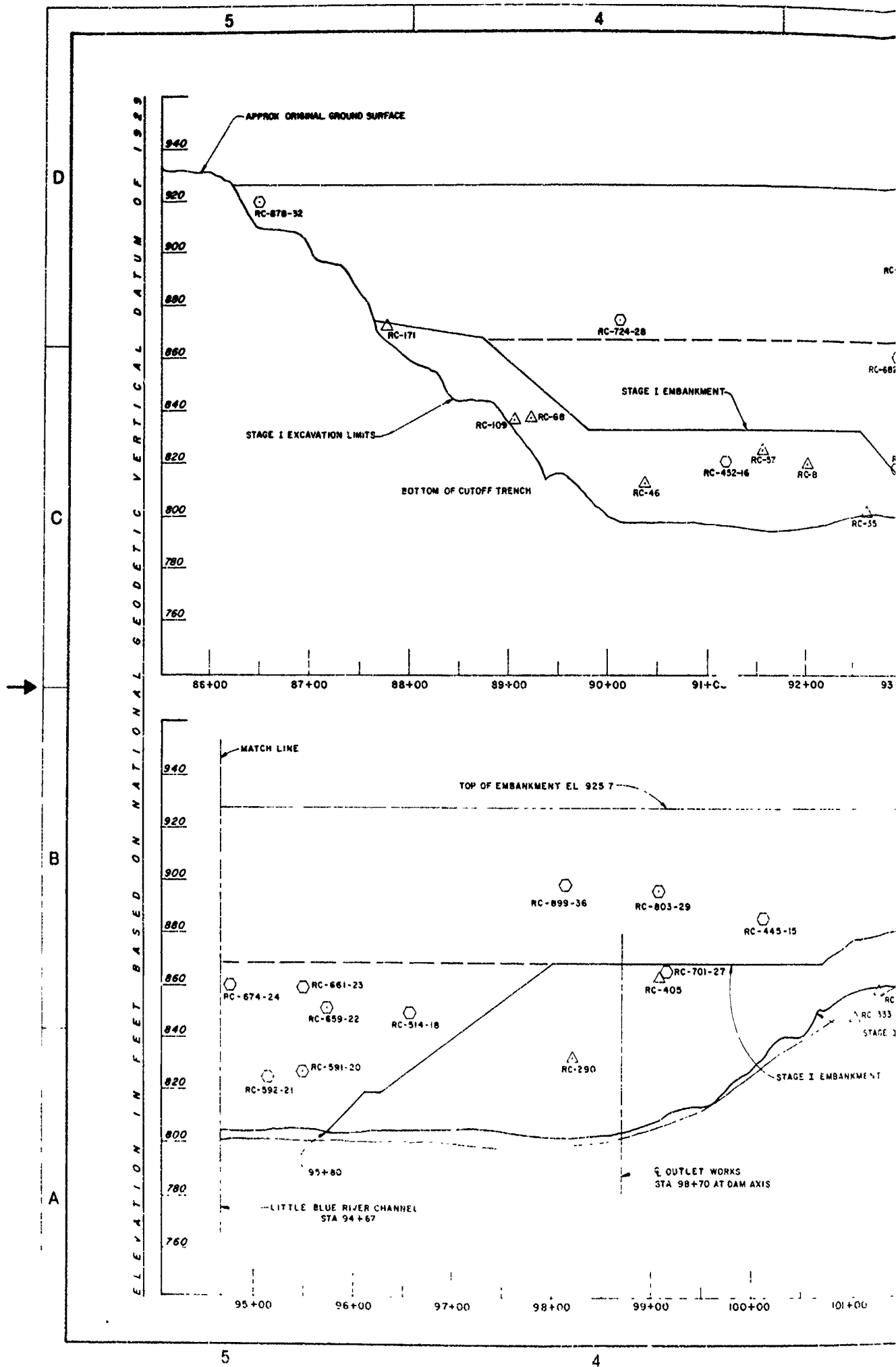
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Date:    
Job No:    
File No: RBL-3-1446

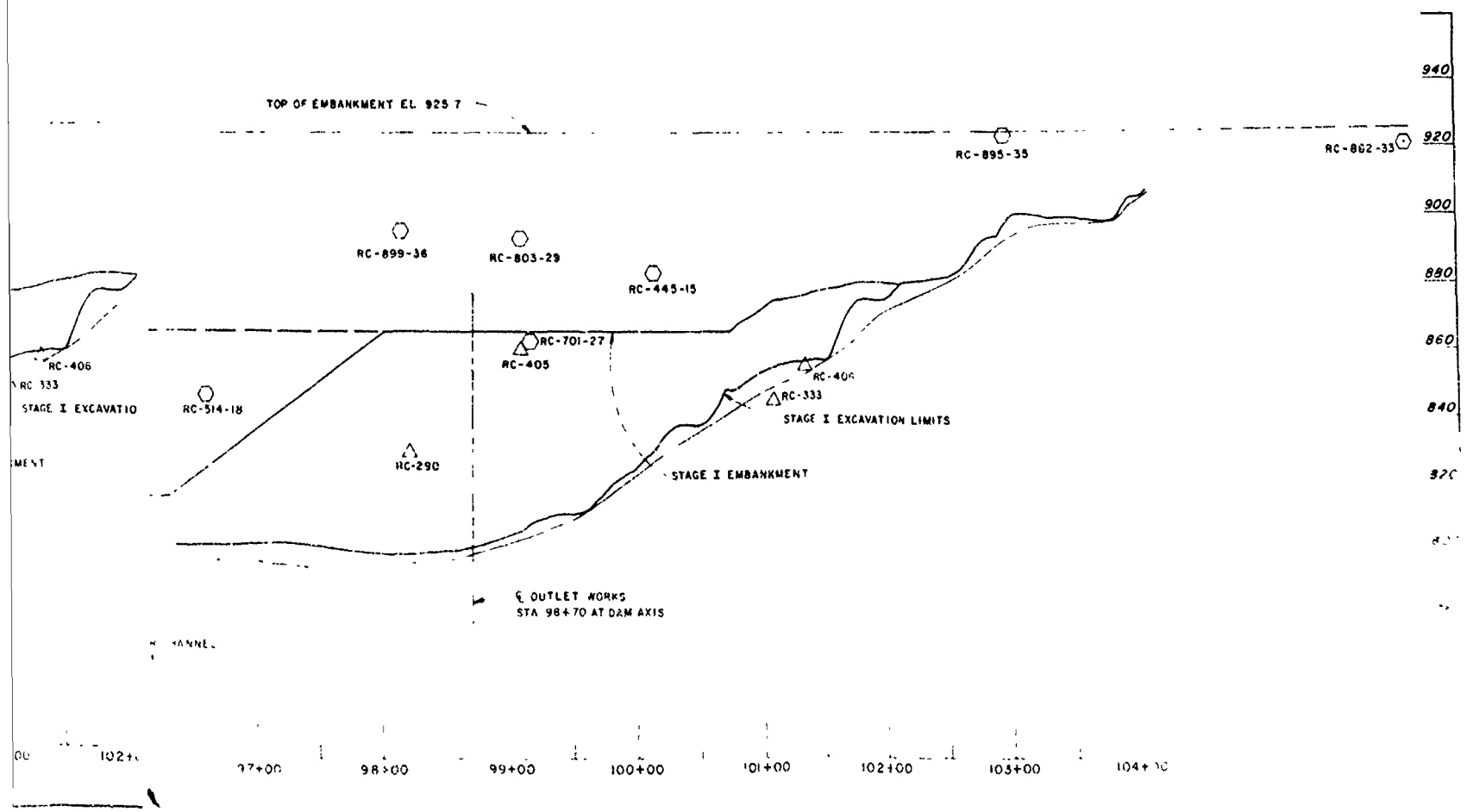
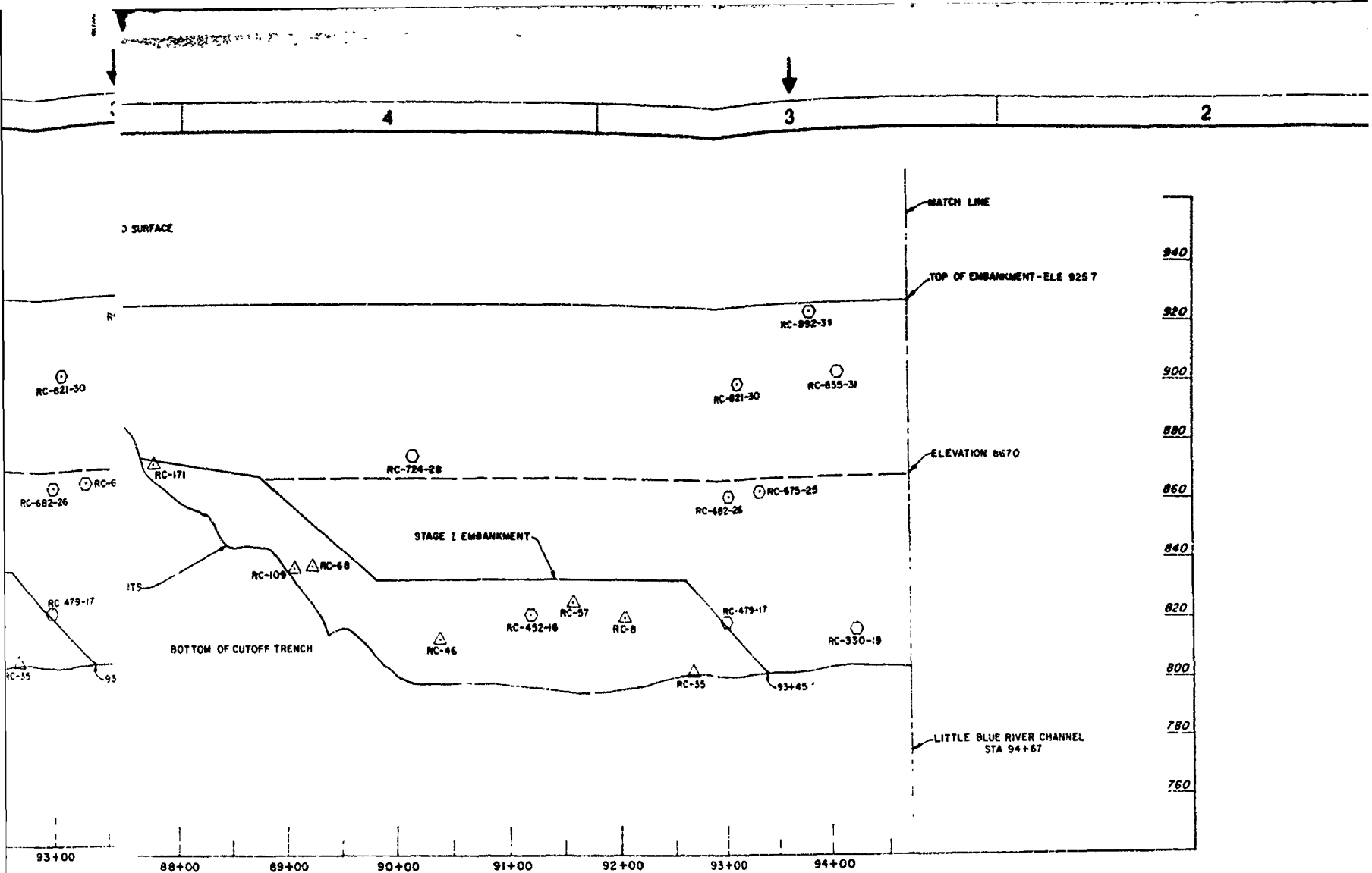


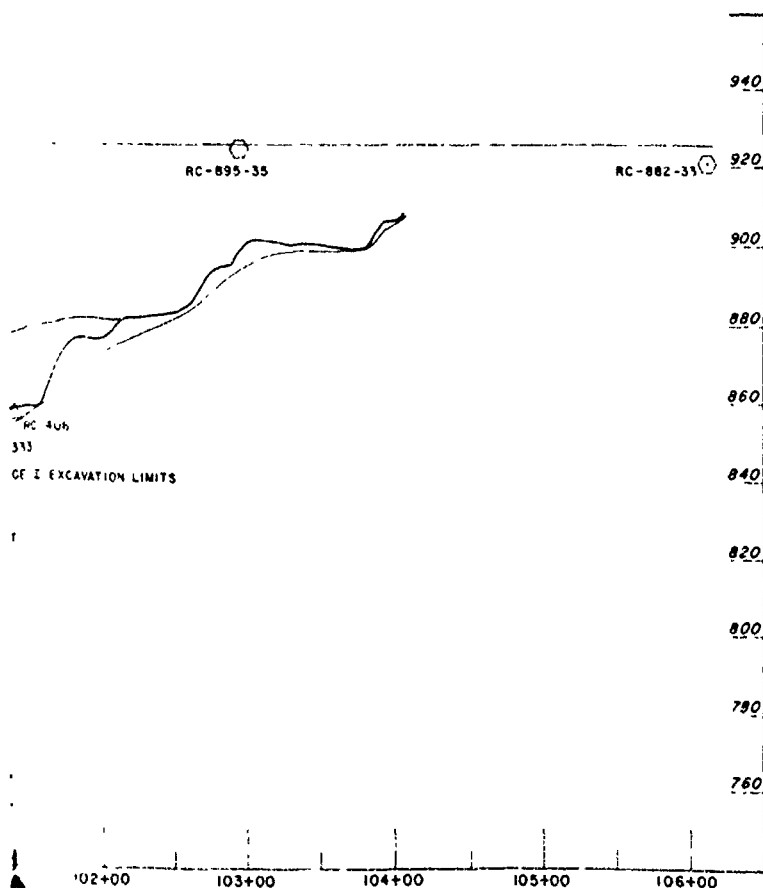
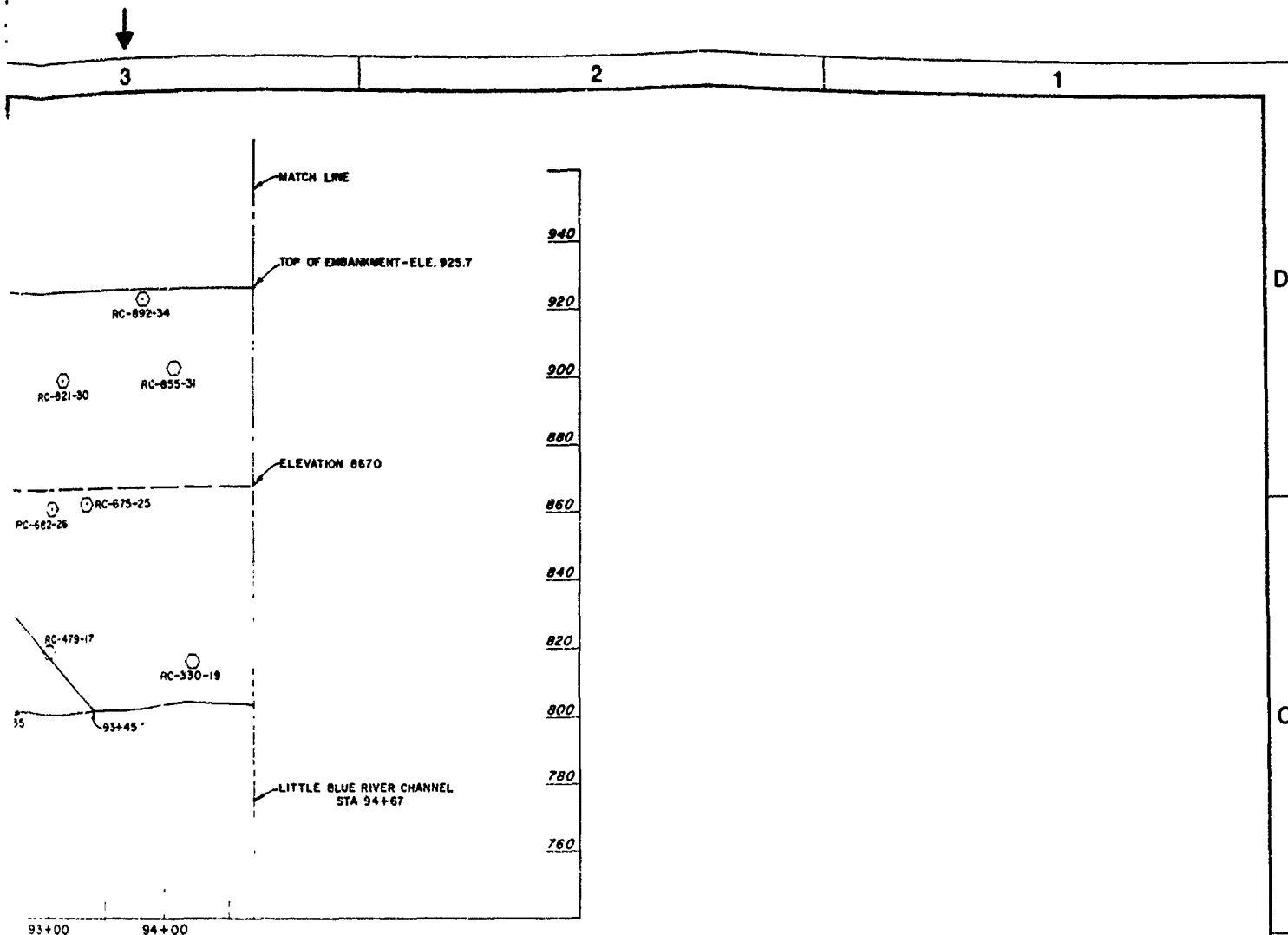












### LEGEND

- △ - STAGE I RECORD CONTROL SAMPLE LOCATION
- - STAGE II RECORD CONTROL SAMPLE LOCATION

Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: WBE

DRAWN BY: EGM

CHECKED BY: WBE/RGD

SUBMITTED BY: FCW

DATE: AS SHOWN

FILE NO: RBL-3-1448

PLATE NO 48







LIQUID LIMIT - 55 IMPERVIOUS ZONE					
STAGE I					
SAMPLE	STATION	RANGE	ELEV.	SAMPLE NO.	SAMPLE DEPTH
RC-35	92+85	0+06 1/2	805.5		
RC-46	90+35	0+15 1/4	817.0		
RC-57	91+55	0+15 1/4	827.0		
RC-68	89+20	0+10 1/2	836.5		
RC-100	89+20	0+30 1/2	835.0		
RC-122	44+50	0+15 1/4	815.0		
RC-171	37+75	0+08 1/2	873.0		
RC-197	51+25	0+16 1/2	811.0		
RC-290	98+20	2+50 1/2	832.0		
RC-333	101+05	2+20 1/2	847.5		
RC-405	98+05	0	864.0		
RC-408	101+32	0+55 1/2	857.5		
OH-1	98+00	1+45 1/2	853.0	1	17.0
			846.0	3	24.0
			836.0	4	34.0
			830.5	6	39.5
OH-2	98+12	0	847.8	1	19.2
			839.6	3	27.4
			837.1	4	29.9
			833.3	5	33.7
			829.3	6	37.7
			825.1	8	41.9
U-595	88+85	1+16 1/4	864.0	WAX 1	3.0
			855.5	5	11.5
PPF-99-1	98+51	2+20 1/2	822.7	1	3.0
			816.7	4	9.0
			815.2	5	10.5
I-99-1	98+36	2+56 1/2	857.5	4	9.5
			840.0	7	17.0
			824.0	12	27.0
			814.0	19	43.0
			805.5	22	53.0
				26	61.5
PPE-99-7	98+40	0+70 1/2	863	1	3.5
			855	5	11.5
			837	11	29.5
			829	14	37.5
			821.5	18	45.0
			810.5	23	56.0
PPE-99-11	98+35	0+86 1/2		20	
			839.2	14	26.0
			827.7	20	39.5
PPF-99-4	98+38	0+40 1/2	823.0	1	

LIQUID LIMIT - 55 IMPERVIOUS ZONE			
STAGE II			
SAMPLE	STATION	RANGE	ELEV.
RC-530-19	94+25	0+50 1/2	815.0
RC-591-20	95+30	0+65 1/2	828.0
RC-661-23	95+50	0+30 1/2	857.0
RC-682-26	93+00	0+25 1/2	862.0
RC-701-27	99+30	1+85 1/2	865.0

LIQUID LIMIT - 60 RANDOM ZONE			
STAGE I			
SAMPLE	STATION	RANGE	ELEV.
RC-8	92+00	1+50 1/2	821.7

LIQU  
STAC  
SAM  
RC-E  
RC-F  
U-7  
U-7

LIQUID LIMIT - 60 IMPERVIOUS ZONE STAGE II					
SAMPLE	STATION	RANGE	ELEV	SAMPLE NO	SAMPLE DEPTH
RC-888-36	98+00	1+50 %	896.0		
RC-808-28	99+00	0+90 %	891.0		
U-746	98+40	1+70 %	828-827.5	4	70.6-71.1
			828.3-825.8	5	72.3-72.8
U-747	99+00	0	921.0-920.5	2	34-39

LIQUID LIMIT - 65 RANDOM ZONE STAGE II					
SAMPLE	STATION	RANGE	ELEV	SAMPLE NO	SAMPLE DEPTH
RC-659-22	95+70	2+60 %	849.0		
RC-582-21	95+10	1+10 %	825.5		
RC-452-16	91+20	0+60 %	822.0		
PPE-92-1	92+00	1+00 %	841.7		58.3
PPE-95-1	94+70	1+00 %	862.2	8	18.8-19.3
			852.0	12	28.8-29.3
PPE-93-12	93+00	1+15 %	835.7	3	54-59

LIQ  
STA  
SA  
RC-  
RC-  
PPE

LIQUID LIMIT - 60 RANDOM ZONE STAGE II					
SAMPLE	STATION	RANGE	ELEV	SAMPLE NO	SAMPLE DEPTH
RC-479-17	93+00	1+80 %	818.0		
RC-674-24	94+75	1+10 %	861.5		
PPE-94-1	94+00	1+40 %	842.5-842.1	3	55-59
			836.5-836.1	6	11.5-11.9
			823.5-823.0	13	24.5-25.0

LIQUID LIMIT - 70 RANDOM ZONE STAGE II					
SAMPLE	STATION	RANGE	ELEV	SAMPLE NO	SAMPLE DEPTH
RC-855-31	94+00	0+30 %	908.0		
RC-877	86+50	0+22 %	919.5		
RC	102+90	0+90 %	924.0		
PPE-92-1	92+00	1+00 %	898.7	1	13-1.8
			893.7	3	6.3-6.8
			861.2	16	38.8-39.3
PPE-95-1	94+70	1+00 %	880.5	1	0-0.5
			869.7	5	11.3-11.6
U-744	94+00	0+30 %	917.4-916.9	3	6.4-6.9
			913.3-912.8	5	10.5-11.0
			903.7-903.2	7	20.1-20.6

LIQUID LIMIT - 65 IMPERVIOUS ZONE STAGE II					
SAMPLE	STATION	RANGE	ELEV	SAMPLE NO	SAMPLE DEPTH
RC-445-15	100+10	2+30 %	886.0		
RC-724-28	90+20	0+05 %	874.0		
RC-821-30	93+20	0+10 %	900.0		
RC-882-35	106+15	0+08 %	921.0		
RC-892-34	93+75	0+10 %	923.5		
U-745	91+00	0	918.0-917.5	3	6.4-6.9
			908.0-907.5	7	16.4-16.9

\* INSPECTION TRENCH

BERM ZONE - NO SPECIFIED LIQUID LIMIT STAGE II					
SAMPLE	STATION	RANGE	ELEV	SAMPLE NO	SAMPLE DEPTH
RC-675-25	93+30	2+20 %	864.0		
RC-514-18	96+56	2+35 %	853.5		
PPE-93-10A	93+00	3+00 %	825.4-824.9	2	2.0-2.5
U-594	89+60	2+80 %	884	1	3.0
			873	6	13.0

SACK #4 COMPOSITE SAMPLE FROM SPENCER SEEP AREA

Revisions			
Symbol	Descriptions	Date	Approved
U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	ROG	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by	PST	RECORD CONTROL SAMPLE SCHEDULE	
Checked by	ROG/WBE	Scale	AS SHOWN
Submitted by	ROG	Sheet number	1 of 1
		Date	
		File No	RBL-3-1450

PLATE NO 50

D

C

B

A

SAMPLE	DEPTH (FT)	NO. OF GRADATION TESTS	TRIAXIAL 'Q' TEST	TRIAXIAL 'R' TEST	TRIAXIAL 'R' TEST	DIRECT SHEAR 'S' TEST	DENSITY TEST	COMPACTION TEST	PERMEABILITY TEST
STAGE I, LIQUID LIMIT - 55 IMPERVIOUS ZONE									
RC-35	2	X	X			X		X	
RC-46	2	X	X			X		X	
RC-57	2	X	X			X		X	
RC-68	2	X	X			X		X	
RC-109	2	X	X			X		X	
RC-122	2	X	X			X		X	
RC-171	2	X	X			X		X	
RC-197	2	X	X			X		X	
RC-290	2	X	X			X		X	
RC-333	2	X	X			X		X	
RC-405	3	X	X			X		X	
RC-406	1	X	X			X		X	
OH-1	17.0		X	X		X			
	24.0		X	X	X	X			
	34.0		X	X	X	X			
	39.5		X	X	X	X			
OH-2	19.2		X	X	X	X			
	27.4	1		X	X	X		X	
	29.9			X	X	X			
	33.7			X	X	X		X	
	37.7		X	X	X	X		X	
	41.9		X	X	X	X		X	
I-99-1	9.5					X			
	17.0	1	X						
	27.0	1				X			
	43.0	1	X	X	X				
	53.0	1				X			
	61.5		X	X	X				
PPF-99-1	3.0	1				X			
	9.0	1				X			
	10.5	1				X			
PPF-99-4	1					X			
PPE-99-7	3.5					X			
	11.5					X			
	29.5	1				X			
	37.5		X	X	X				
	45.0					X			
	56.0		X	X					
PPE-99-11	29	1				X			
	39.5	2				X			
U-595	3.0	1		X	X				
	11.5	1		X	X				

SAMPLE	DEPTH (FT)	NO. OF GRADATION TESTS	TRIAXIAL 'Q' TEST	TRIAXIAL 'R' TEST	TRIAXIAL 'R' TEST
STAGE II, LIQUID LIMIT - 55 IMPERVIOUS ZONE					
RC-530-19	3	X	X	X	
RC-581-20	1	X	X	X	
RC-661-23	2				X
RC-662-26	2	X			X
RC-701-27	2				X
STAGE I, LIQUID LIMIT - 60 RANDOM ZONE					
RC-8	2	X	X		X
STAGE II, LIQUID LIMIT - 60 IMPERVIOUS ZONE					
RC-803-29	2	X	X	X	
RC-899-36	1				
U-746	70.8-71.1	1	X	X	X
	72.3-72.8	1	X	X	X
U-747	34-39	1	X	X	X
STAGE II, LIQUID LIMIT - 60 RANDOM ZONE					
RC-479-17	2	X	X	X	
RC-674-24	2	X	X	X	
PPE-94-1	5.5-5.9	1	X	X	X
	11.5-11.9	1	X	X	X
	24.5-25.0	1	X	X	X
STAGE II, LIQUID LIMIT - 65 IMPERVIOUS ZONE					
RC-445-15	1	X	X	X	X
RC-724-28	2	X	X	X	
RC-821-30	2	X	X	X	
RC-882-33	2	X	X	X	
RC-892-34	1				
U-745	6.4-6.9	2	X	X	X
	16.4-16.9	1	X		

SAMPLE	DEPTH (FT)	NO OF								
		TRIAxIAL			TRIAxIAL			TRIAxIAL		DIRECT
STAGE II, LIQUID LIMIT - 55 IMPERVIOUS ZONE										
RC-830-19		3	X	X	X				X	X
RC-881-20		1	X	X	X				X	X
RC-881-23		2						X	X	X
RC-882-26		2	X						X	X
RC-701-27		2						X	X	X
STAGE I, LIQUID LIMIT - 60 RANDOM ZONE										
RC-8		2	X	X				X		X
STAGE II, LIQUID LIMIT - 60 IMPERVIOUS ZONE										
RC-803-29		2	X	X	X				X	X
RC-889-36		1								X
U-746	70.8-71.1	1	X	X	X				X	X
	72.3-72.8	1	X	X	X				X	X
U-747	34-39	1	X	X	X					
STAGE II, LIQUID LIMIT - 60 RANDOM ZONE										
RC-475-17		2	X	X	X				X	X
RC-674-24		2	X	X	X				X	X
PPE-94-1	5.5-5.9	1	X	X	X				X	
	11.5-11.9	1	X	X	X					
	24.5-25.0	1	X	X	X					
STAGE II, LIQUID LIMIT - 65 IMPERVIOUS ZONE										
RC-445-15		1	X	X	X	X	X	X	X	X
RC-724-28		2	X	X	X				X	X
RC-921-30		2	X	X	X				X	X
RC-882-33		2	X						X	X
RC-692-34		1								X
U-745	6.4-6.9	2	X	X	X					
	6.4-16.9	1	X							

SAMPLE	DEPTH (FT)	NO. OF GRAINS							
		TRIAXIAL	TRIAXIAL	TRIAXIAL	TRIAXIAL	DIRECT STRESS	DEMS		
STAGE II, LIQUID LIMIT - 65 RANDOM ZONE									
RC-452-16		1	X	X	X	X	X	X	
RC-592-21		2	X	X	X			X	X
RC-659-22		2	X	X	X			X	X
PPE-93-12	54-59	1	X	X	X			X	
PPE-95-1	18.8-19.3	1					X		
	28.8-29.3	1	X	X	X				
STAGE II, LIQUID LIMIT - 70 RANDOM ZONE									
RC-855-31		2	X	X	X			X	X
RC-878-32		2	X	X	X			X	X
RC-896-33		1							X
PPE-92-1	1.3-1.8	1	X	X	X				
	6.3-6.8	1	X	X	X				
	13.8-15.0	1					X		
	38.8-39.3	1	X	X	X				
PPE-95-1	0.0-0.5	1	X	X	X				
	11.3-11.8	1	X	X	X				
U-744	6.4-6.9	1	X	X	X				
	10.5-11.0	1	X	X	X				
	20.1-20.4	1	X	X	X				
STAGE II, BERM ZONE (UNSPECIFIED LIQUID LIM.T)									
RC-514-18		2	X	X	X			X	X
RC-675-25		2	X	X	X			X	X
PPE-93-10A		1	X						
U-594	3.0	-		X	X				
	13.0	-	X	X	X				
SACK #1		1	X	X	X				X

Revisions			
Symbol	Descriptions	Date	Approved


**U S ARMY ENGINEER DISTRICT**  
**CORPS OF ENGINEERS**  
**KANSAS CITY MISSOURI**

Designed by  
**RDG**

Drawn by  
**PST**

Checked by  
**RDG / WBE**

Submitted by  
**RDG**



**U.S. Army Corps  
of Engineers**

**LITTLE BLUE RIVER, MISSOURI**  
**LONGVIEW LAKE**  
**EMBANKMENT CRITERIA**  
**AND PERFORMANCE REPORT**

**LABORATORY SOILS TESTING LOG**

Soil No.	AS SHOWN	Sheet Number
Date		
49		

1 of 1      RBL-3-1451

DATA ON RECORD CONTROL SAMPLES													
STAGE I LIQUID LIMIT - 55 IMPERVIOUS ZONE													
HOLE NO	SAMPLE DEPTH	CLASSIFICATION				DRY DENSITY		WATER CONTENT			"Q" TEST		"R" TAN $\delta$
		SYM	L L	P.L	P.I.	MAX.	INITIAL RANGE	OPT	INITIAL RANGE	PERCENT SATURATION	TAN $\phi$	C T.S.F	
RC-35		CH	65	22	43	97.2	96.8 - 97.1	23.5	26.6 - 27.4	97-99	0.052	1.08	
RC-48		CL	40	18	22	105.2	106.4 - 107.5	16.5	19.9 - 20.2	97-98	0.096	1.00	
RC-57		CL	48	16	32	105.0	107.6 - 108.3	17.8	18.5 - 18.6	88-89			
RC-68		CH	50	20	30	97.4	98.5 - 98.7	21.2	24.0 - 24.1	91-92	0.070	1.25	
RC-109		CL	48	18	28	103.0	107.3 - 107.8	19.0	18.9 - 19.1	90	0.016	2.15	
RC-122		CL	44	18	26	102.2	106.5 - 107.5	18.7	20.2 - 20.7	95	0	1.35	
RC-171		CL	46	19	27	103.8	99.3 - 101.0	18.8	24.8 - 25.9	100	0.035	1.25	
RC-197		CH	57	17	40	99.7	93.1 - 93.3	21.3	29.3 - 29.5	100	0.123	0.55	
RC-290		CL	43	20	23	105.3	104.6 - 104.9	18.7	21.6 - 21.9	99-100	0.044	1.70	
RC-333		CL	45	25	20	101.6	97.5 - 97.8	20.3	25.7 - 25.9	100	0.121	0.75	
RC-405		CH	56	22	34	95.6	99.0 - 100.7	24.3	23.6	92-96	0.155	1.23	
RC-406		CH	56	20	36	97.7	105.7 - 106.1	22.2	20.6 - 20.8	94-95	0.070	2.38	
OH-1	17.0	CH	53	16	37		104.8 - 105.7		21.6 - 22.0	100			0.521
	24.0	CL	43	15	28		107.2 - 108.1		20.1 - 20.4	98-100	0.035	1.35	0.613
	34.0	CL	41	16	25		99.5 - 103.6		21.5 - 23.8	92-95			0.53
	39.5	CL	40	16	24		102.8 - 104.4		20.5 - 22.2	93-97	0.070	0.40	0.53
OH-2	19.2	CL	48	16	32		105.7 - 107.9		18.9 - 19.7	92-98	0.167	1.15	0.46
	27.4	CL	36	18	18		106.6 - 107.3		18.3 - 18.7				0.58
	29.9	CL	39	18	21		106.9 - 107.8		19.4 - 20.1	93-97			0.57
	33.7	CL	41	16	25		110.2 - 110.9		18.8 - 19.0	100			0.57
	37.7	CL	32	16	16		113.0 - 115.0		16.4 - 17.1	98-99	0.456	1.45	0.68
	41.9	CL	43	16	27		102.0 - 102.3		22.7 - 23.0	98-100	0.437	0.75	0.57
U-595	3.0	CH	53	17	36		100.8 - 101.8		23.1 - 23.4	94-95			0.48
	11.5	CL	45	17	28		108.6		17.5	86			
PPF-99-1	3.0						91.2 - 94.5		26.0 - 27.1	89-97			
	9.0						100.8 - 105.4		19.8 - 21.3	90-94			
	10.5						102.4 - 104.4		21.7	94-99			
I-99-1	9.5						102.3 - 105.1		21.3 - 21.9	93-98	0.035	1.50	
	17.0	CL	46	17	29		101.1 - 103.9		22.3 - 24.0	97-98			
	27.0						103.2 - 107.8		18.5 - 21.0	90-93			
	43.0	CL	41	15	25		106.5 - 107.4		19.8 - 20.4	95-99	0.100	0.75	0.60
	53.0						103.9 - 104.3		23.8 - 23.9	100			
	61.5	CL	41	14	27		102.4 - 110.3		18.2 - 23.1	96-100	0.315	0.40	0.41
PPF-99-4							99.1 - 103.4		21.5 - 22.0	92-96			0.41
PPC-99-7	3.5						100.0 - 100.7		22.1 - 22.5	89-90			
	11.5						100.0 - 101.6		23.3 - 24.2	92-99			
	29.5						106.4 - 106.7		18.9 - 19.1	89-96			
	37.5	CL	33	17	16		104.9 - 107.3		20.3 - 21.4	95-96			0.5
	45.0						104.7 - 105.0		21.8	97-98			
	56.0	CL	48	17	31		104.2 - 105.4		19.8 - 21.0	96-98			

NOTE: EXCEPT FOR THE "R" TEST, ALL SHEAR STRENGTH PARAMETERS PROVIDED ON THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT TANGENT TO THE "R" TEST-EFFECTIVE STRESS STRENGTH PARAMETERS WERE DERIVED FROM THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING MOHR C DIAGRAM

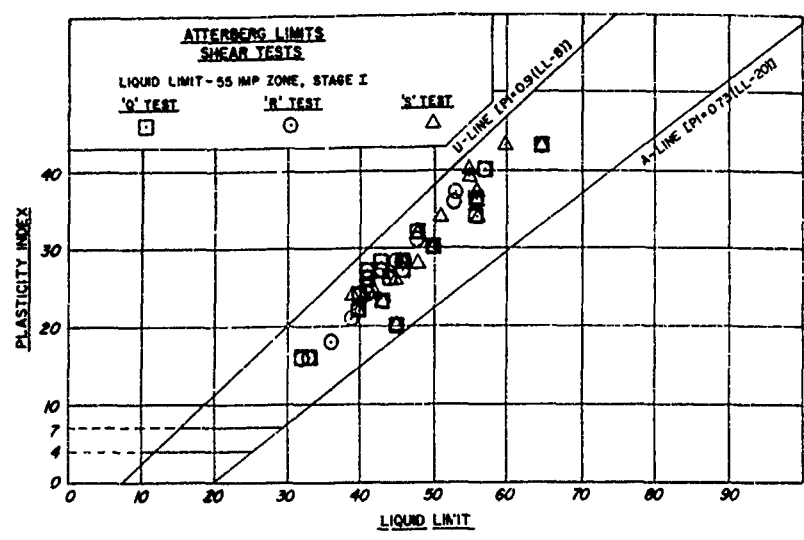
\* SUSPECT TEST

LIQUID LIMIT - 55 IMPERVIOUS ZONE

1 INL  
DIAGRAM

WERE DERIVED FROM A BEST-FIT TANGENT TO THE MOHR'S CIRCLES FOR EACH SAMPLE TESTED.  
IS DESIGNATED BY THE CORRESPONDING MOHR'S DIAGRAM.

PLATE NO 52



LIQUID LIMIT - 55  
IMPERVIOUS ZONE  
STAGE I

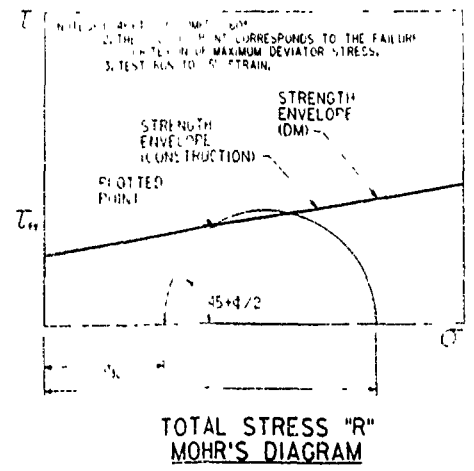
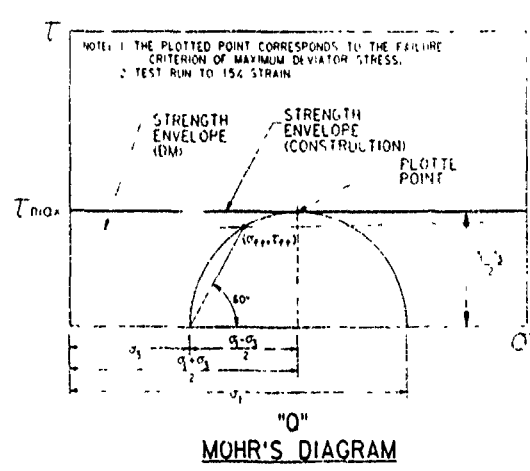
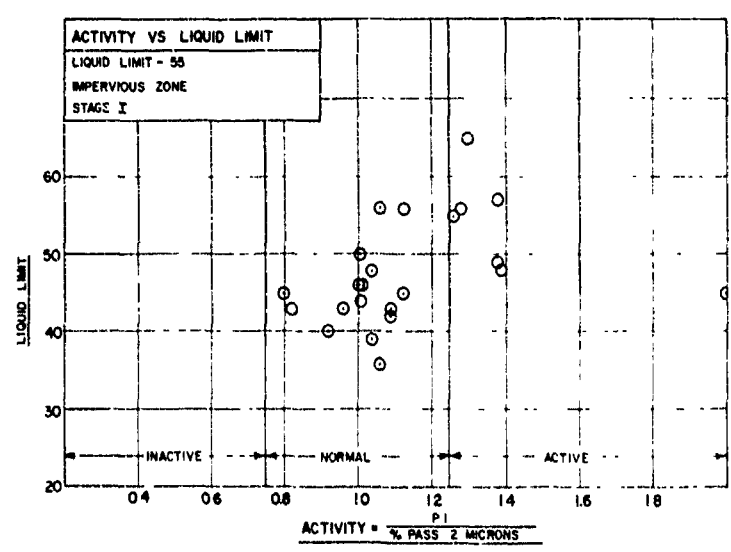
SAMPLE

OH-2

DIFFERENTIAL HEAD -  
HORIZONTAL SPECIMEN

MATI

BERM  
HILLSIDE BO  
VALLEY BOF  
FOUNDATION  
SHALE - PE  
SHALE - RE





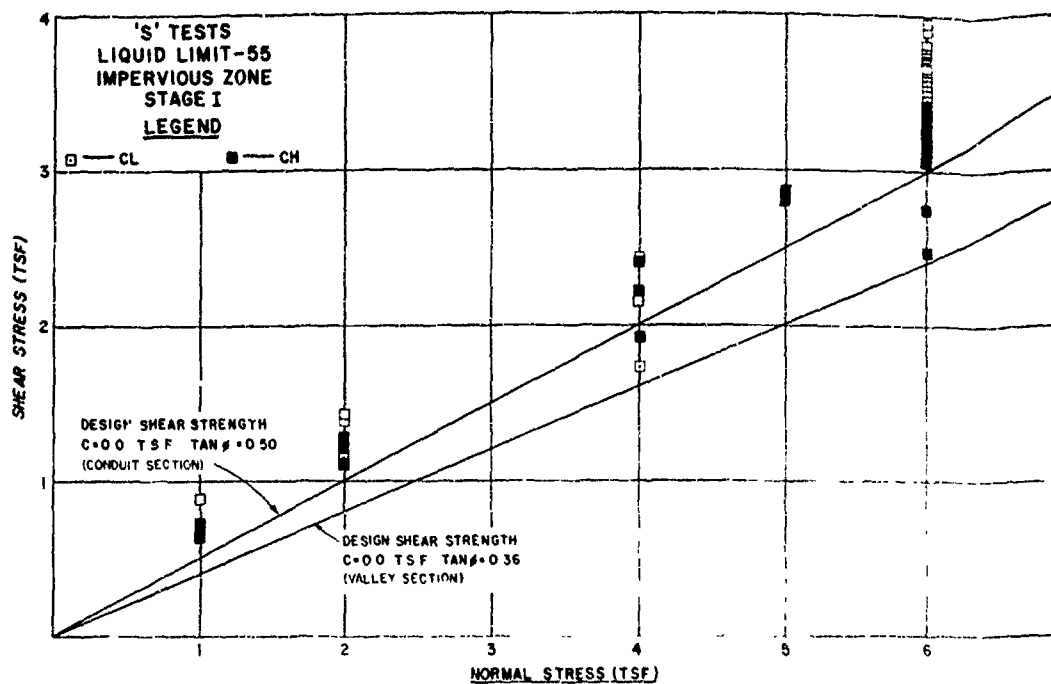


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4

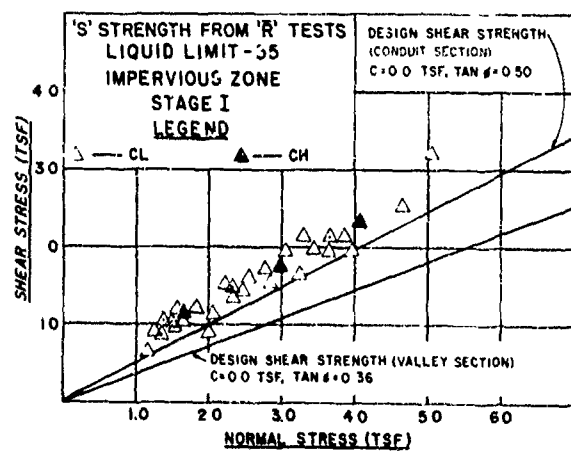
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D

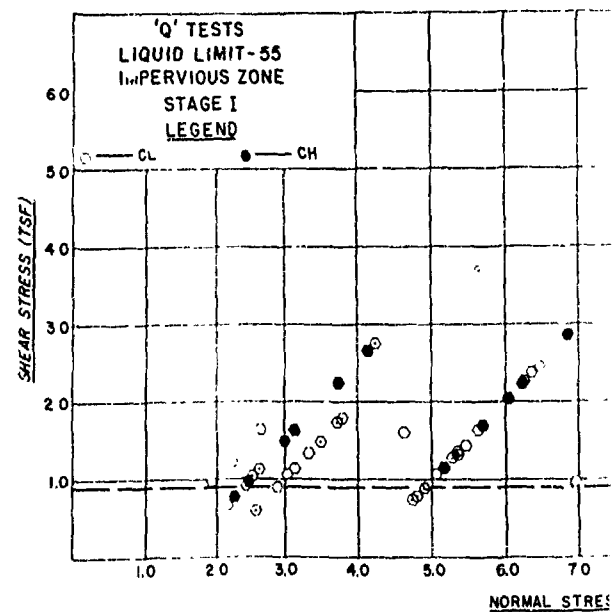


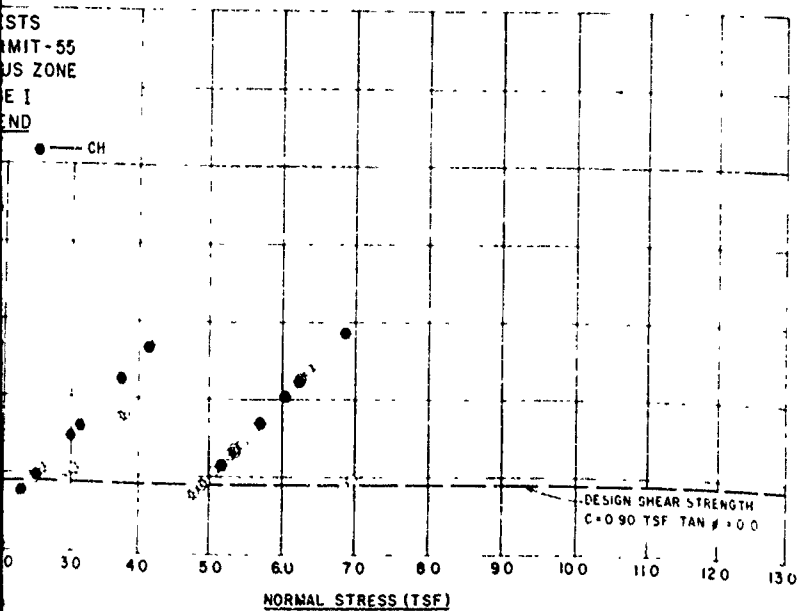
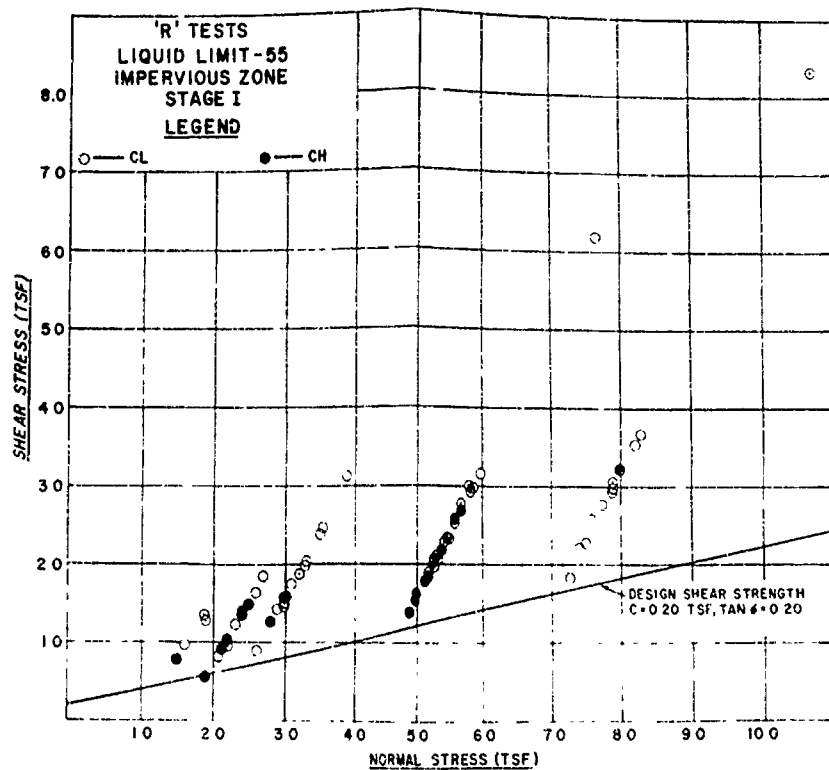
C

B



A





Revisions		Date	Approved
Symbol	Descriptions		

U S ARMY ENGINEER DISTRICT  
 CORPS OF ENGINEERS  
 KANSAS CITY, MISSOURI

Design by: RDG  
 Drawn by: MAW  
 Checked by: RDG / WBE  
 Submitted by: RDG

**LITTLE BLUE RIVER, MISSOURI**  
**LONGVIEW LAKE**  
**EMBANKMENT CRITERIA AND**  
**PERFORMANCE REPORT**  
**SHEAR TEST SUMMARIES**  
**LIQUID LIMIT-55 IMPERVIOUS ZONE**  
**STAGE I**

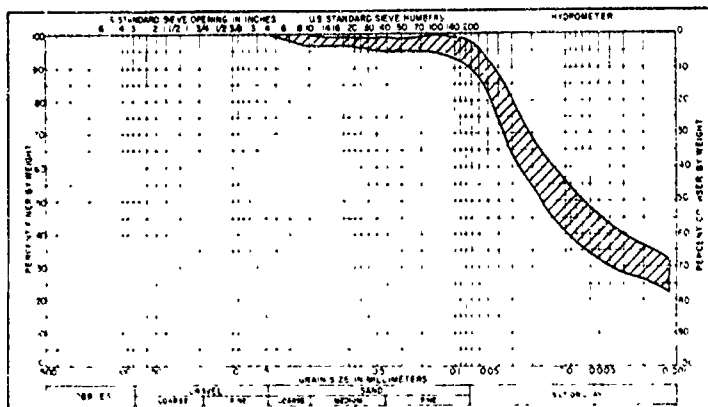
AS SHOWN

Sheet number: \_\_\_\_\_  
 Date: \_\_\_\_\_  
 File No: **RBL-3-1454**

# **DATA ON RECORD CONTROL SAMPLES** **STAGE II LIQUID LIMIT -55 IMPERVIOUS ZONE**

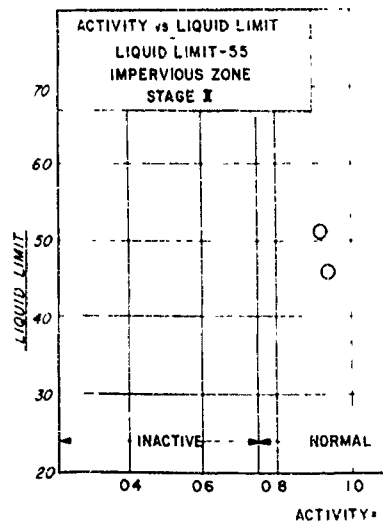
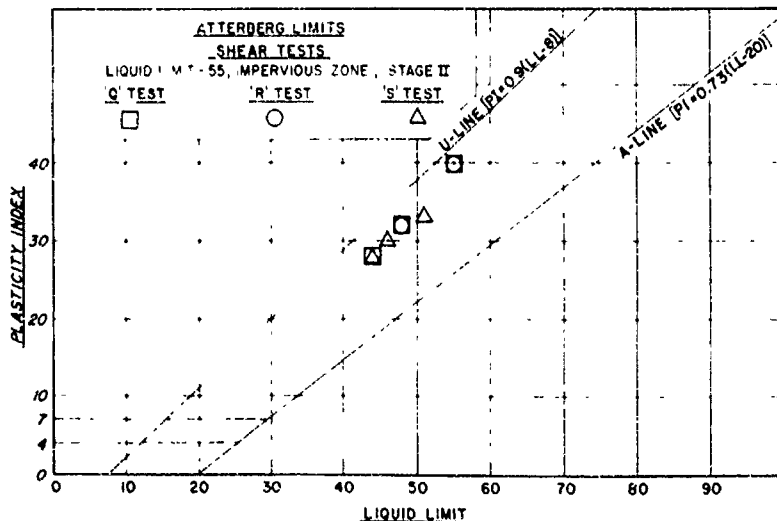
HOLE NO.	SAMPLE DEPTH	CLASSIFICATION				DRY DENSITY		WATER CONTENT		PERCENT SATURATION	"Q" TEST		"R" TEST	
		SYM.	L.L.	P.L.	P.I.	MAX.	INITIAL RANGE	OPT.	INITIAL RANGE		TAN $\phi$	C T.S.F.	TAN $\phi$	C T.S.F.
RC-530-19		CL	48	16	32	100.6	103.0 - 104.8	20.6	20.7 - 21.9	94-96	0.087	1.05	0.532	0.19
RC-59-20		CH	55	15	40	98.3	102.5 - 103.8	22.1	22.6 - 23.0	100	0.018	0.85	0.404	0.21
RC-66-23		CL	46	16	30	98.1		22.7						
RC-682-26		CL	44	16	28	100.5	101.1 - 106.4	19.8	18.6 - 19.0	77-87	0.298	1.15		
RC-701-27		CH	51	18	33	98.3		22.5						

NOTE: EXCEPT FOR THE "R" TEST, ALL SHEAR STRENGTH PARAMETERS PROVIDED ON THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT TANGENT TO THE MOHR'S CIRCLES. THE "R" TEST-EFFECTIVE STRESS STRENGTH PARAMETERS WERE DERIVED FROM THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING MOHR'S DIAGRAM.



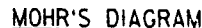
LIQUID LIMIT - 55 IMPERVIOUS ZONE STAGE II  
 GRADATION LIMITS FOR RECORD CONTROL SAMPLES TESTED.

PHYSICAL SOIL				
MATERIAL	UNIT WEIGHT P.C.F.	SAT. MOIST (C.T.S.F.)	WATER CONTENT (%)	PLASTICITY INDEX
BERM	125	140	0.45	0.00
HILLSIDE BORROW	125	145	0.90	0.00
VALLEY BORROW	125	145	0.90	0.00
FOUNDATION OVERBURDEN	115	140	0.60	0.00
SHALE - PEAK	140	140		
SHALE - RESIDUAL	140	140		



ERVIOUS ZONE

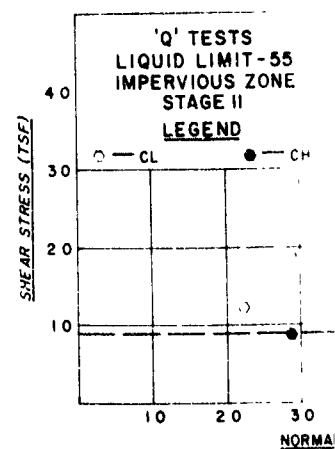
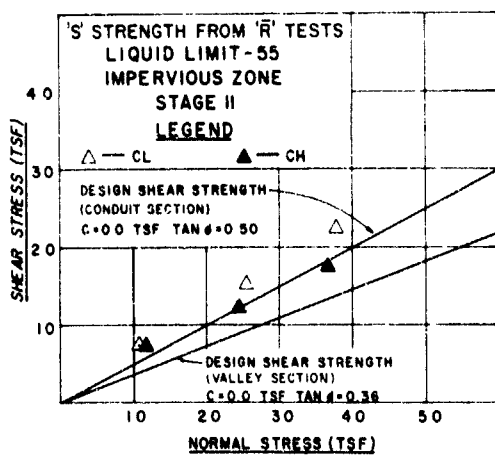
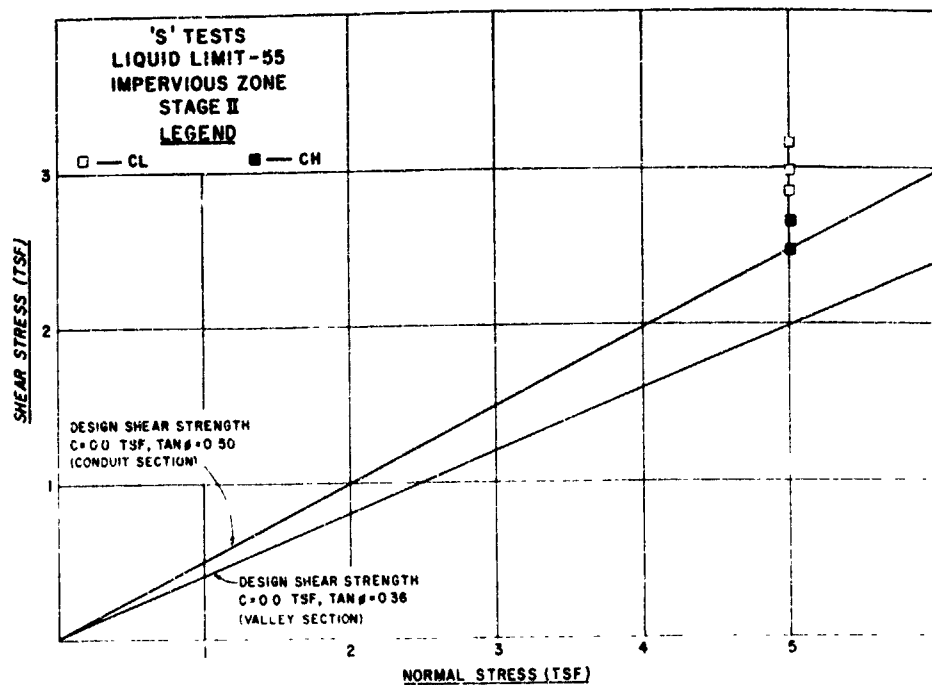
VED FROM A BEST FIT TANGENT TO THE MOHR'S CIRCLES FOR EACH SAMPLE TESTED.  
BY THE CORRESPONDING MOHR'S DIAGRAM.



TOTAL STRESS "R"  
MOHR'S DIAGRAM



PLATE NO 55

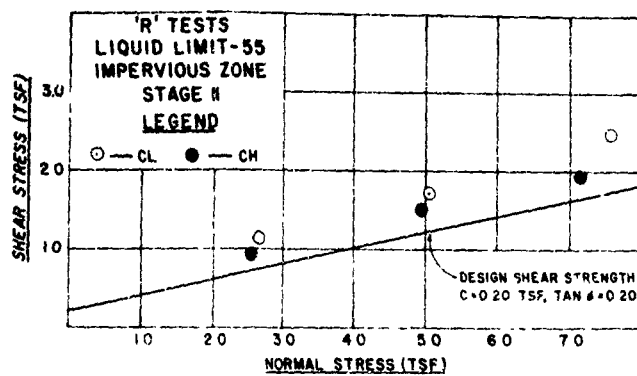


3

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1

D



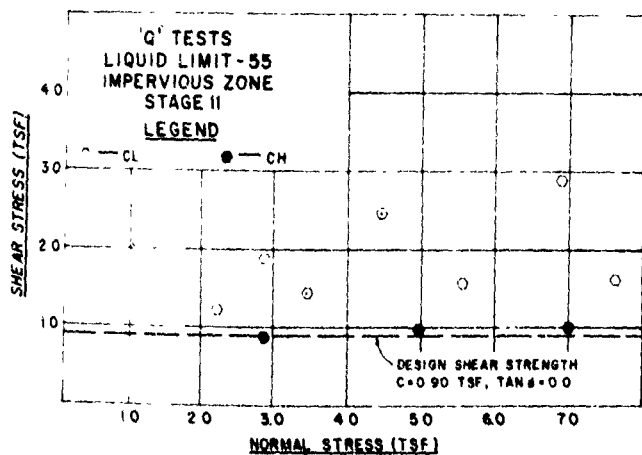
C

B

A

35  
VE

CH

30  
NORMAL

Revisions		Date	Approved
Symbol	Descriptions		
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	WBE	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by	MAW	SHEAR TEST SUMMARIES LIQUID LIMIT-55 IMPERVIOUS ZONE STAGE II	
Checked by	WBE/RGD	Scale	AS SHOWN
Submitted by	FCW	Date	
		Sheet number	
		File No.	RBL-3-1456

PLATE NO. 56

3

2

1

D

C

B

A

5

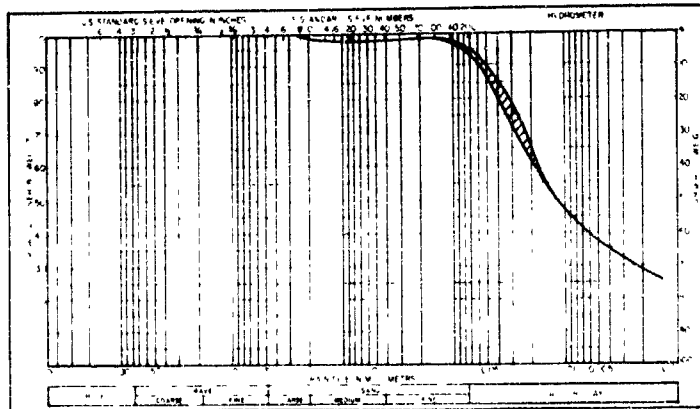
4

3

# DATA ON RECORD CONTROL SAMPLES STAGE I LIQUID LIMIT - 60 RANDOM ZONE

HOLE	SAMPLE	CLASSIFICATION				DRY DENSITY		WATER CONTENT			
NO	DEPTH	SYM	L.L.	P.L.	PI	MAX	INITIAL RANGE	OPT	INITIAL RANGE	PER CENT SATURATION	TA
RC-8		CH	51	15	36	100.8	99.8 - 103.7	20.4	22.3 - 24.6	98	0.0

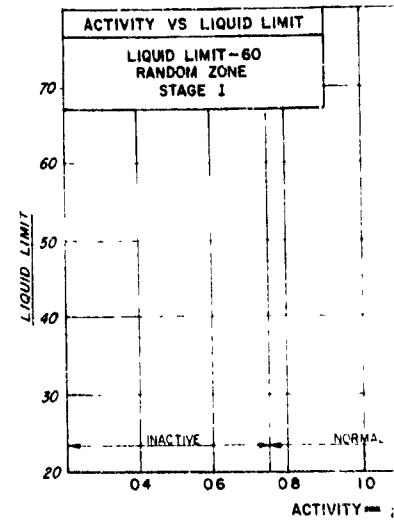
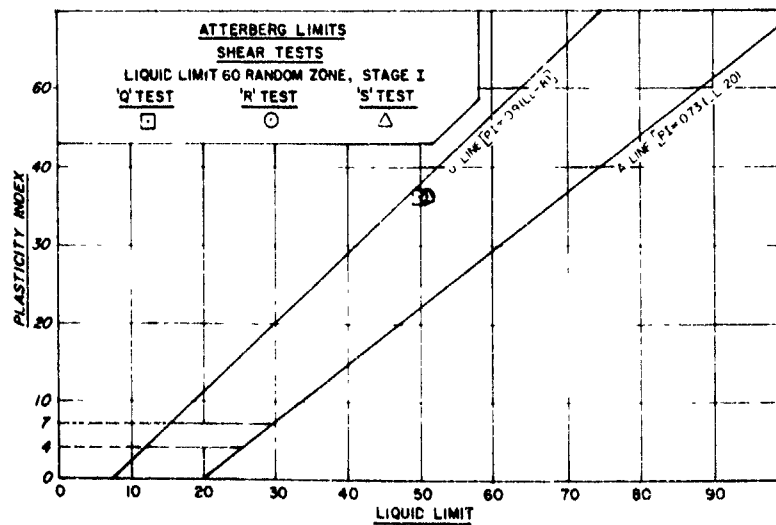
NOTE: ALL "Q" AND "R" SHEAR STRENGTH PARAMETERS PROVIDED ON THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT LINE TANGENT TO THE MOHR'S CIRCLES FOR EACH SAMPLE.



LIQUID LIMIT - 60 RANDOM ZONE STAGE I  
GRADATION LIMITS FOR RECORD CONTROL SAMPLES TESTED.

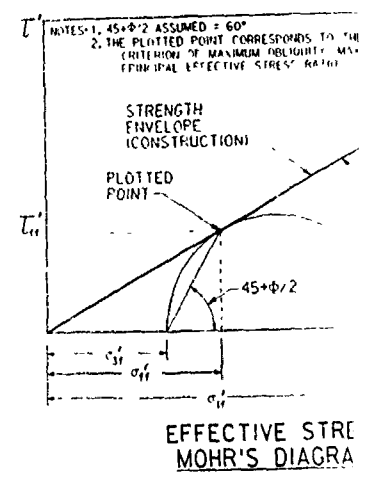
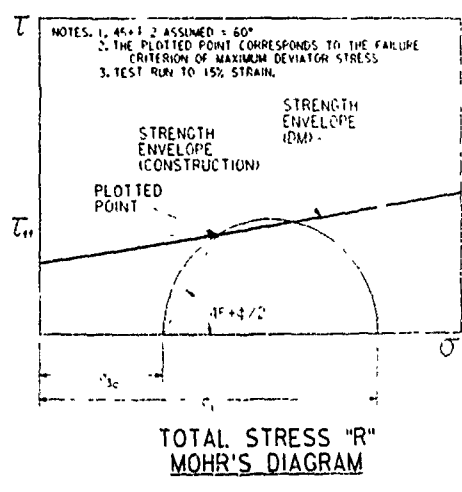
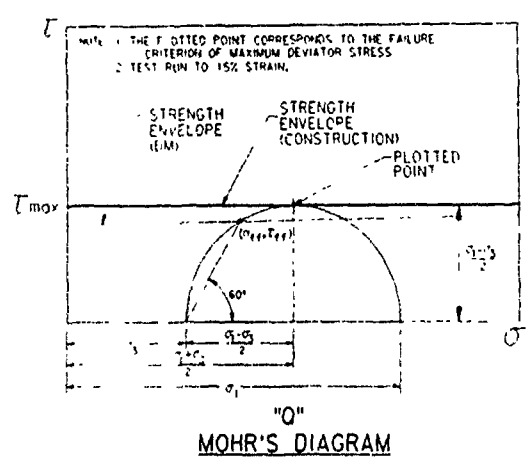
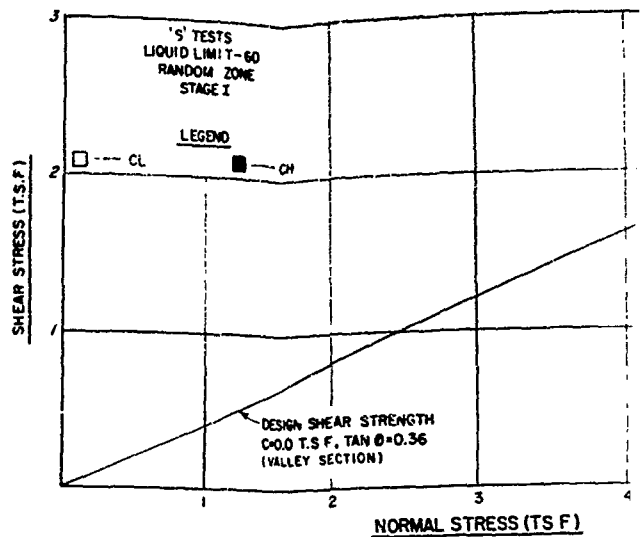
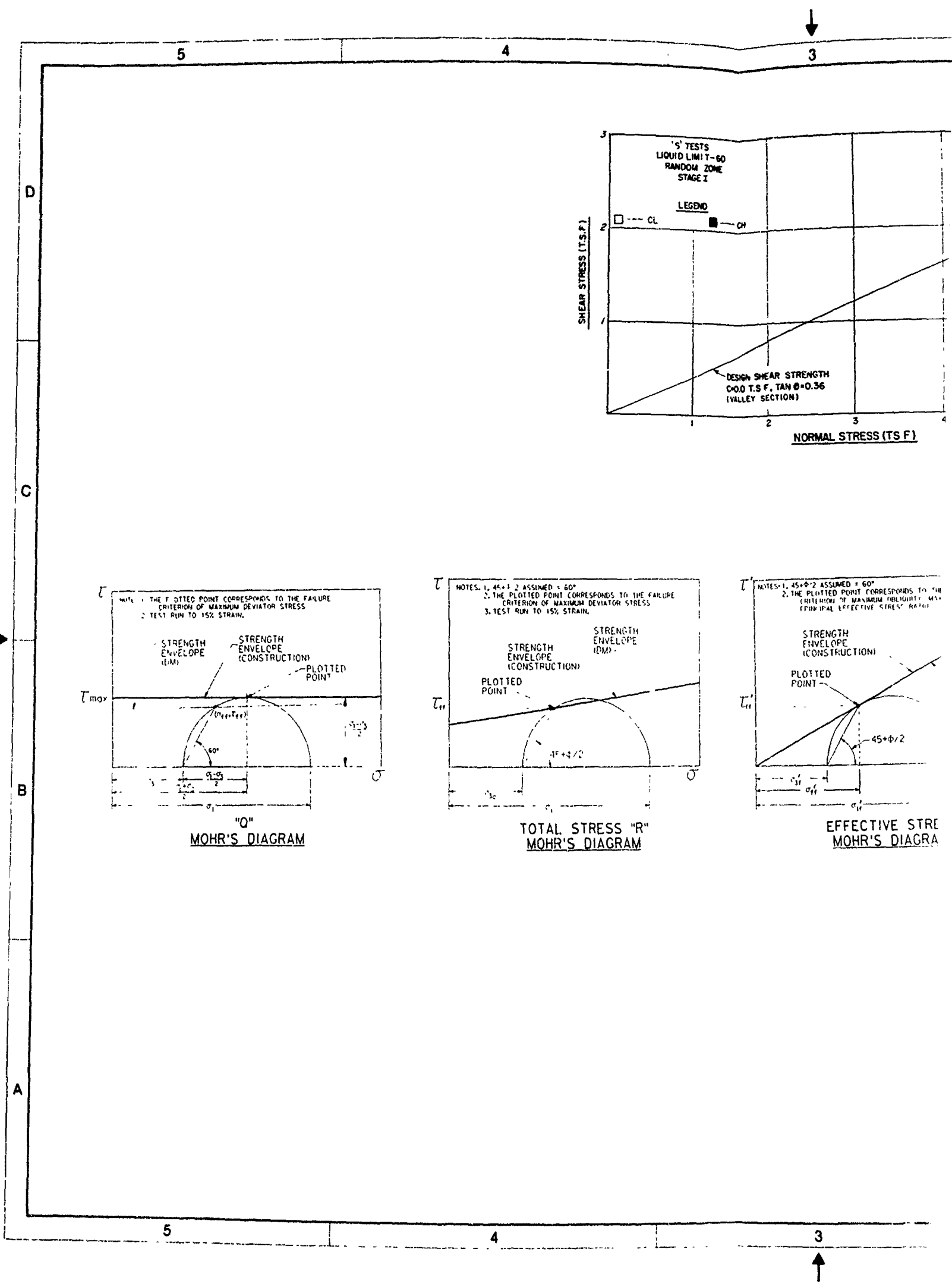
## PHYSICAL SOIL C

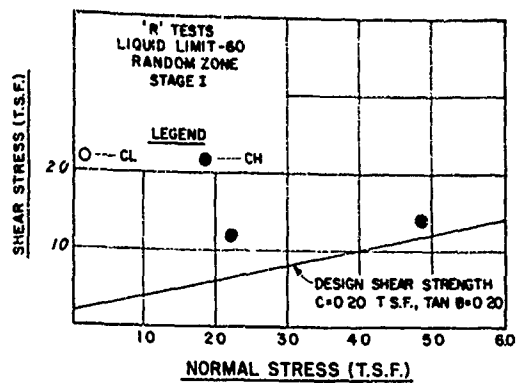
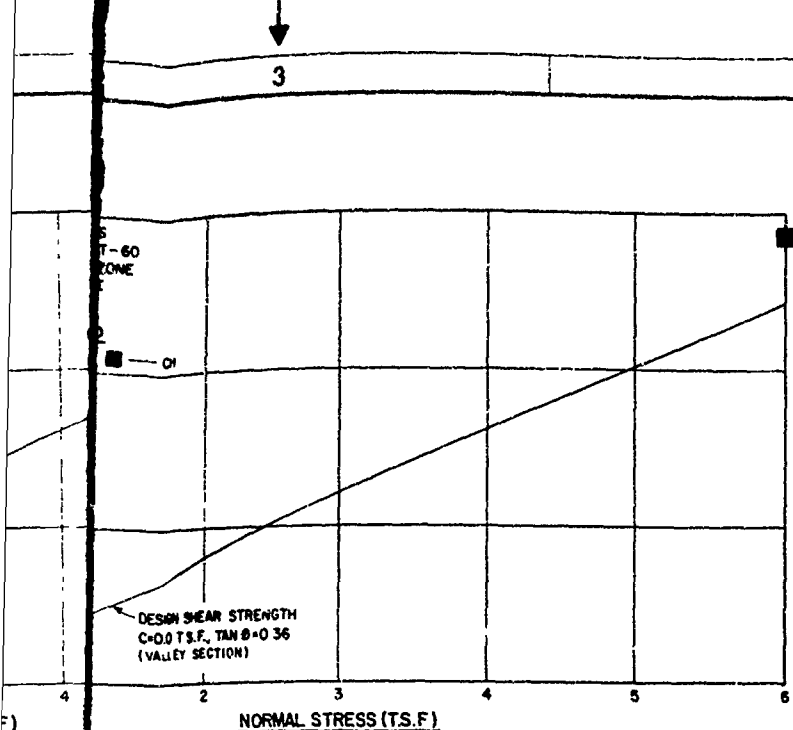
MATERIAL	UNIT WEIGHT		O	T
	SAT. MOIST	P.C.F.		
BERM	115	110	0.45	0.00
HILLSIDE BORROW	125	115	0.90	0.00
VALLEY BORROW	125	115	0.90	0.00
FOUNDATION OVERBURDEN	115	110	0.60	0.00
SHALE - PEAK	140	140	-	-
SHALE - RESIDUAL	140	140	-	-



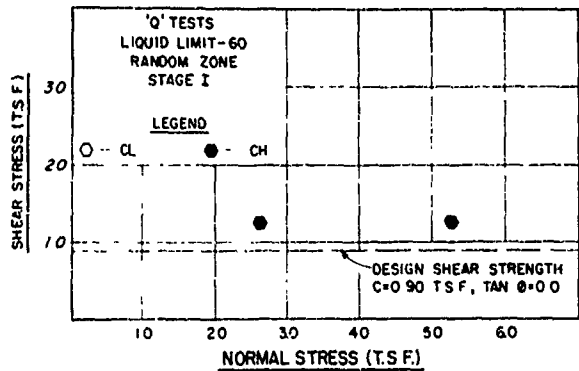
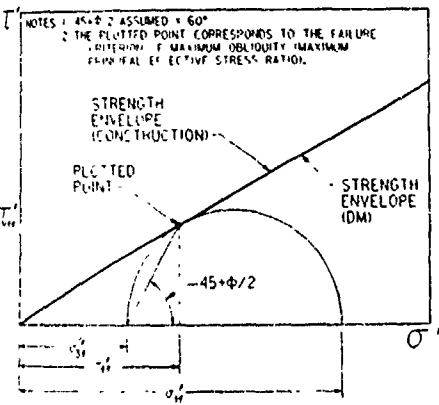








NOTES TO THE  
DESIGNER



STRESS  
DIAGRAM

Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: RDG

Drawn by: SEY

Checked by: RDG/WBE

Submitted by: RDG

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
SHEAR TEST DATA SUMMARIES  
LIQUID LIMIT-60 RANDOM ZONE  
STAGE I

Scale: AS SHOWN

Date:  

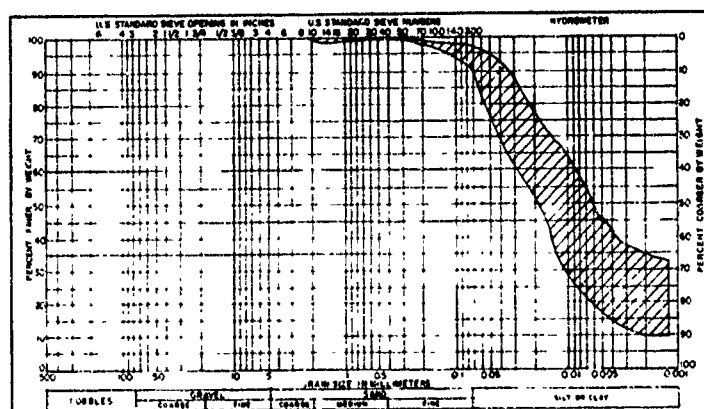
Sheet number:  

File No: RBL-3-1458

D

DATA ON RECORD CONTROL SAMPLES										
STAGE II LIQUID LIMIT - 60 IMPERVIOUS ZONE										
HOLE NO.	SAMPLE DEPTH	CLASSIFICATION				DRY DENSITY		WATER CONTENT		
		SYN.	L.L.	P.L.	P.I.	MAX.	INITIAL RANGE	OPT.	INITIAL RANGE	PERCENT SATURATION
RC-80369		CH	53	24	29	98.0	101.3 - 103.6	23.2	22.9 - 24.4	100
U-746	70.8-71.1	CL	43	18	25		105.5 - 106.3		20.8 - 21.4	97-98
	72.3-72.8	CL	48	20	25		101.1 - 101.4		23.2 - 23.5	97
U-747	34-39	CH	67	16	51		103.5 - 105.2		21.3 - 22.3	93-95

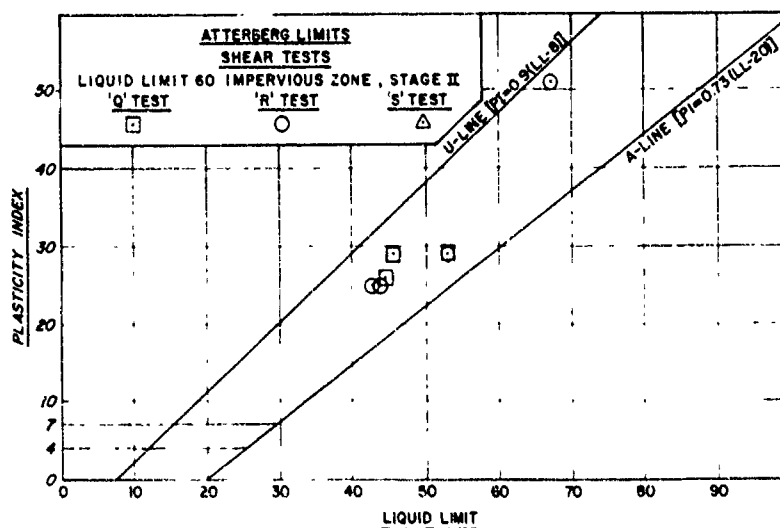
NOTE: EXCEPT FOR THE  $\star$  TEST, ALL SHEAR STRENGTH PARAMETERS PROVIDED ON THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT LINE. THE  $\star$  TEST-EFFECTIVE STRESS STRENGTH PARAMETERS WERE DERIVED FROM THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING SYMBOL.



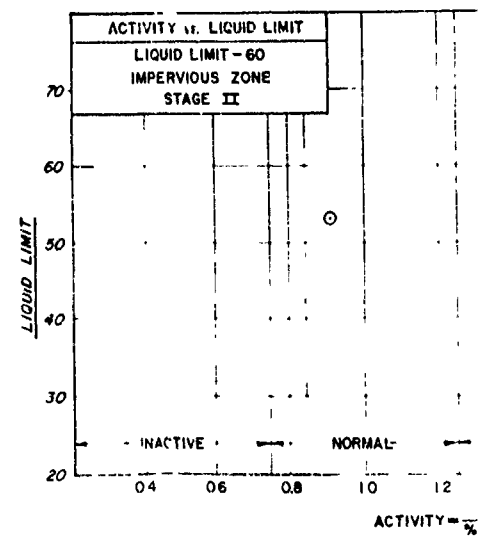
LIQUID LIMIT - 60 IMPERVIOUS ZONE STAGE II  
GRADATION LIMITS FOR RECORD CONTROL SAMPLES TESTED.

PHYSICAL SOIL				
MATERIAL	UNIT WEIGHT P.C.F.	SAT. MOIST	U.T.S.F.	0
BERM	115	110	0.45	0.00
HILLSIDE BORROW	125	115	0.90	0.00
VALLEY BORROW	125	115	0.90	0.00
FOUNDATION OVERCLODEN	115	110	0.60	0.00
SHALE - PEAK	140	140	-	-
SHALE - RESIDUAL	140	140	-	-

B

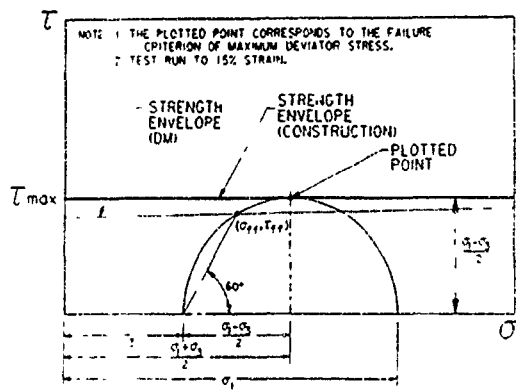


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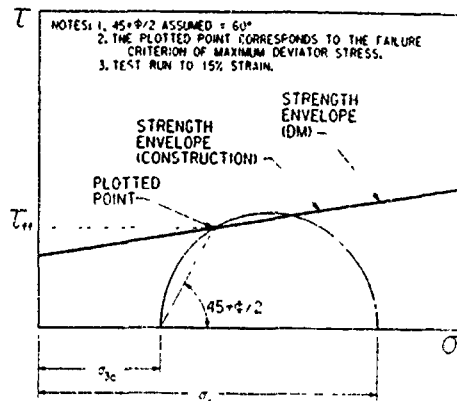




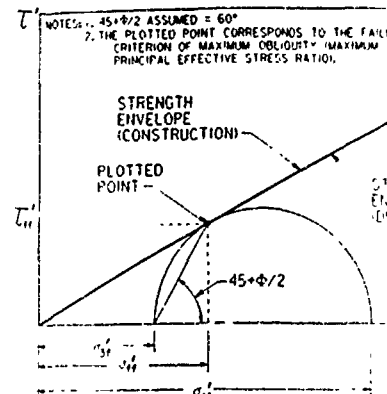
D



"Q" MOHR'S DIAGRAM

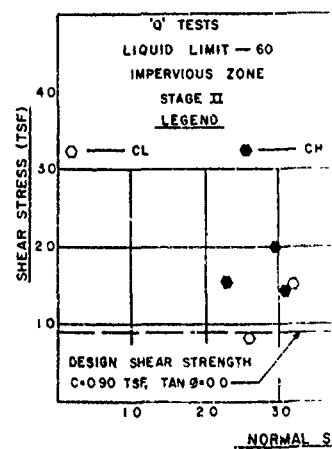
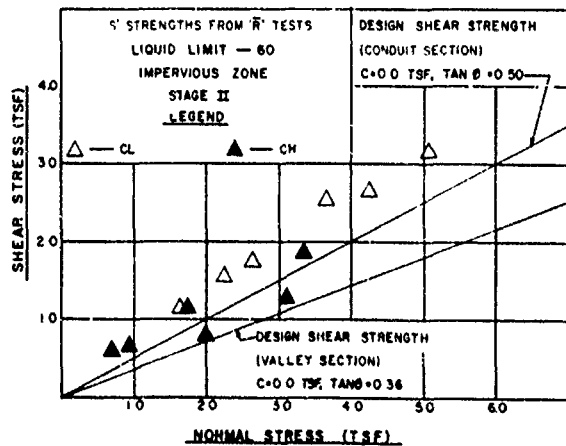


TOTAL STRESS "R" MOHR'S DIAGRAM



EFFECTIVE STRESS MOHR'S DIAGRAM

B

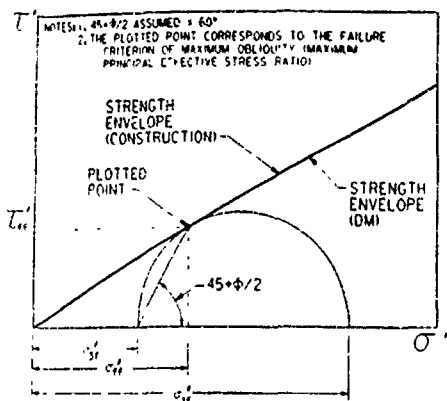
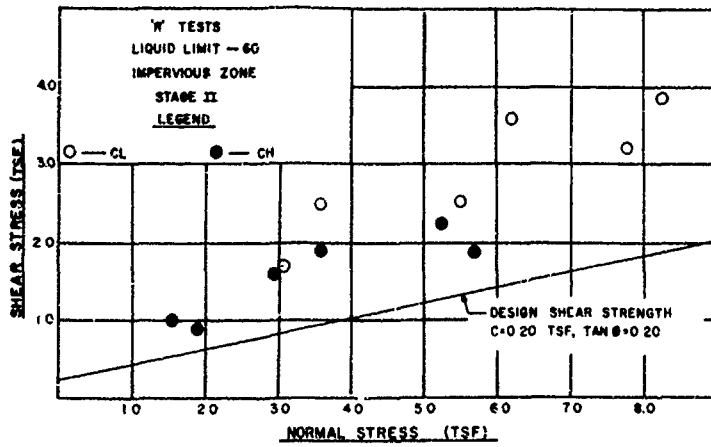


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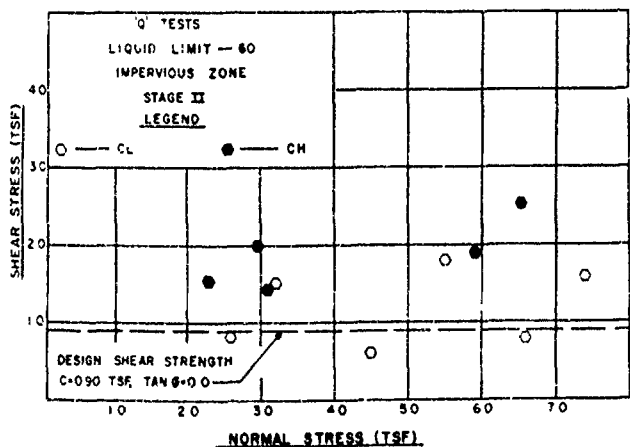
3

2

1



EFFECTIVE STRESS  
MOHR'S DIAGRAM



Revisions			
Symbol	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by WBE	 US Army Corps of Engineers	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT SHEAR TEST SUMMARIES LIQUID LIMIT-60 IMPERVIOUS ZONE STAGE II	
Drawn by JGS		Scale AS SHOWN	Sheet number
Checked by RGD/WBE	Date		
Submitted by FCW	Dwg No.		File No. RBL-3-1460

3

2

1

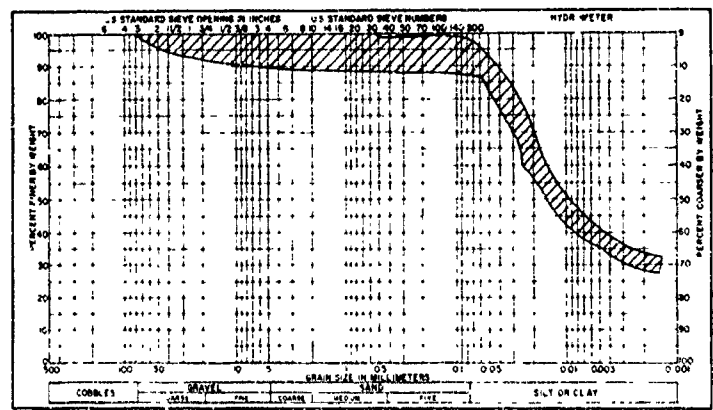
PLATE NO. 60

D  
C  
B  
A

# DATA ON RECORD CONTROL SAMPLES STAGE II LIQUID LIMIT - 60 RANDOM ZONE

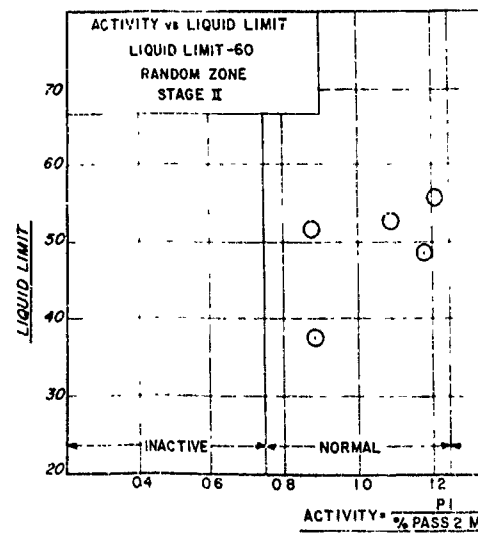
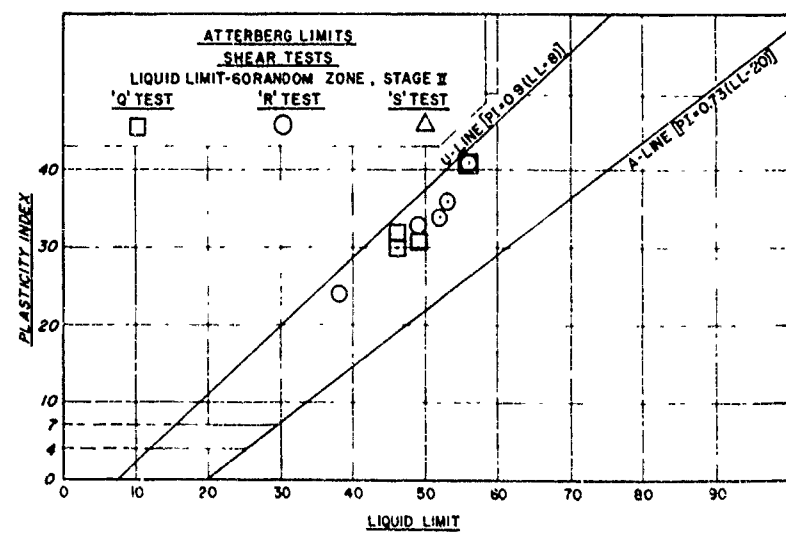
HOLE NO.	SAMPLE DEPTH	CLASSIFICATION				DRY DENSITY		WATER CONTENT				TAN $\phi$
		SYM.	L.L.	P.L.	P.I.	MAX	INITIAL RANGE	OPT	INITIAL RANGE	PERCENT SATURATION		
RC-479-17		CH	52	18	34	97.8	103.4-104.5	22.7	22.2-22.8	100		0.05
RC-674-24		CH	52	15	41	98.0	98.4-99.7	22.8	24.6-25.1	96-97		0.03
PPE-94-1	5.5-5.9	CH	53	17	36		99.6-102.4		22.9-24.6	96		0.024
	11.5-11.9	CL	49	16	33		106.5-108.0		20.4-21.3	100		0.005
	24.5-25.0	CL	38	14	24		107.6-110.6		19.4-20.8	96-100		0.04

NOTE: EXCEPT FOR THE "R" TEST, ALL SHEAR STRENGTH PARAMETERS PROVIDED ON THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT. THE "R" TEST-EFFECTIVE STRESS STRENGTH PARAMETERS WERE DERIVED FROM THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING



LIQUID LIMIT - 60 RANDOM ZONE STAGE II  
GRADATION LIMITS FOR RECORD CONTROL SAMPLES TESTED.

PHYSICAL SOIL CONSTANTS						
MATERIAL	UNIT WEIGHT		DESIGN			
	SAT. MOIST	P.C.F.	CIT. SF	$\phi$	TAN $\phi$	CIT. SF
BERM	15	10	0.45	0.00	0.00	0.1
HILLSIDE BORROW	125	15	0.90	0.00	0.00	0.2
VALLEY BORROW	125	15	0.90	0.00	0.00	0.2
FOUNDATION OVERBURDEN	15	10	0.60	0.00	1.00	0.1
SHALE - PEAK	140	140				
SHALE - RESIDUAL	140	140				



3

2

1

## DATA ON RECORD CONTROL SAMPLES

## PAGE II LIQUID LIMIT-60 RANDOM ZONE

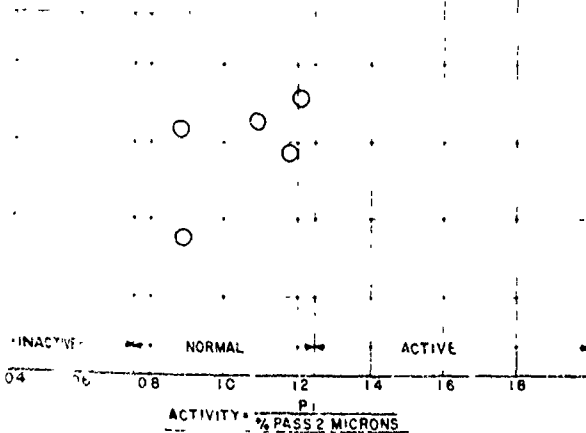
SITY	WATER CONTENT			"Q" TEST		"R" TEST		"R" TEST		"S" TEST DIRECT SHEAR	
	OPT	INITIAL RANGE	PERCENT SATURATION	TAN $\theta$	C T.S.F.	TAN $\theta$	C T.S.F.	TAN $\theta$	C T.S.F.	TAN $\theta$	C T.S.F.
1045	227	22.2-22.8	100	0.052	1.00	0.456	0.20	0.222	0.50		
997	228	24.6-25.1	96-97	0.035	1.03	0.456	0.07	0.240	0.20		
1024		22.9-24.6	96	0.026	1.03	0.414	0.20	0.096	1.25		
1080		20.4-21.3	100	0.009	1.18	0.466	0.28	0.259	0.73		
1106		19.4-20.8	98-100	0.044	0.78	0.577	0	0.335	0		

IN THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT TANGENT TO THE MOHR'S CIRCLES FOR EACH SAMPLE TESTED.  
FROM THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING MOHR'S DIAGRAM.

## PHYSICAL SOIL CONSTANTS

MATERIAL	UNIT WEIGHT P.C.F.		DESIGN SHEAR STRENGTHS									
	SAT. MOIST	Ø	TAN Ø	CIT.S.F.	Ø	TAN Ø	CIT.S.F.	Ø	TAN Ø	CIT.S.F.	Ø	TAN Ø
BORROW	15	95	0.45	0.00	0.00	0.00	11.3	0.20	0.00	16.7	0.30	
FOR	125	95	0.90	0.00	0.00	0.20	11.3	0.20	0.00	19.8	0.36	
NON OVERBURDEN	125	95	0.90	0.00	0.00	0.20	11.3	0.20	0.00	26.6	0.50	
AS	40	40							0.00	31.0	0.60	
QUA	40	40							0.70	16.7	0.30	

## ACTIVITY - LIQUID LIMIT

LIQUID LIMIT -60  
RANDOM ZONE  
STAGE II

ACTIVITY - % PASS 2 MICRONS

Revisions	Date	Approved
Descriptions		
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI		
LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT TEST DATA SUMMARIES LIQUID LIMIT-60 RANDOM ZONE STAGE II		
WBE	AS SHOWN	
RAC		
RGD/WBE		
FCW		
RBL-3-1461		

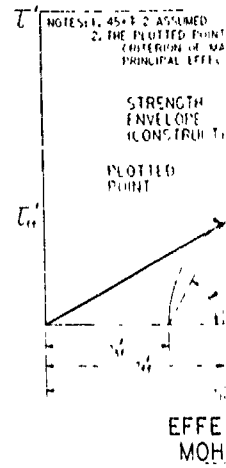
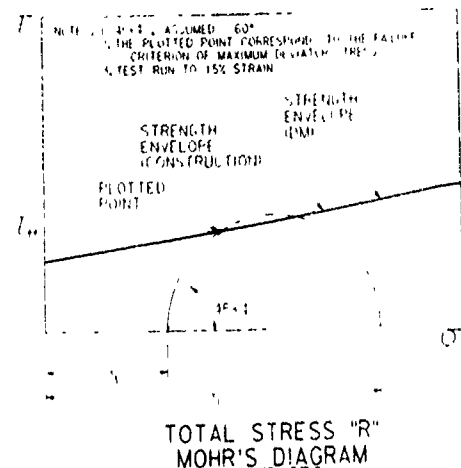
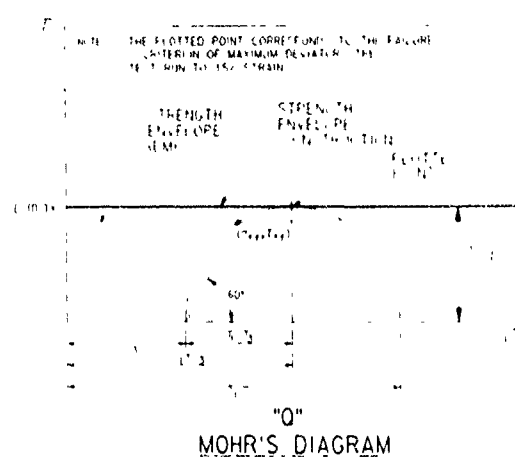
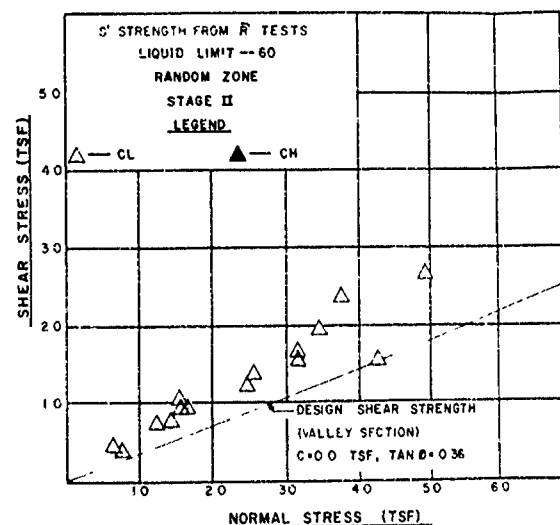
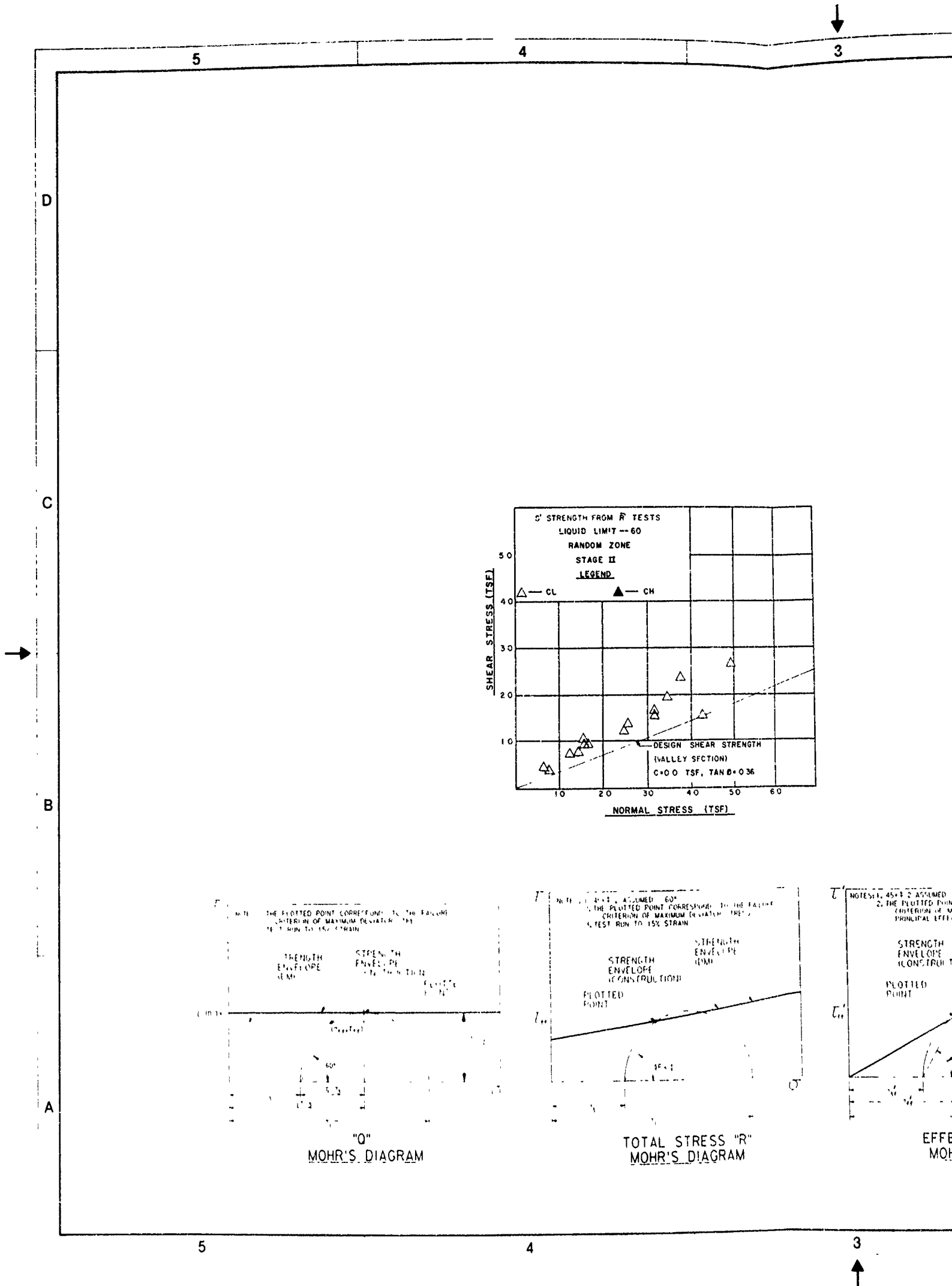
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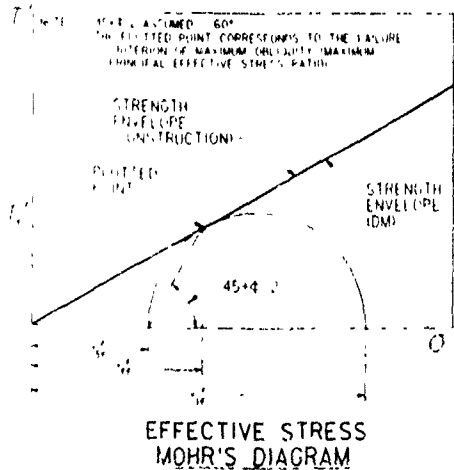
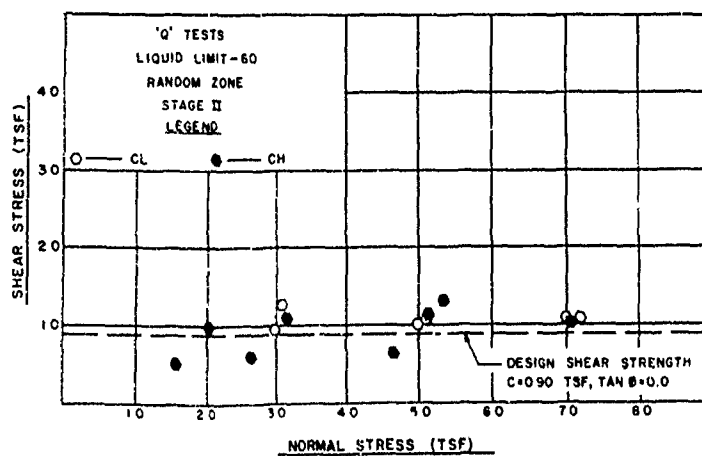
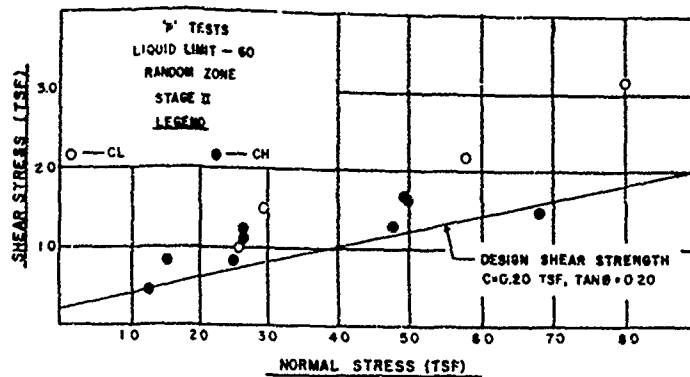
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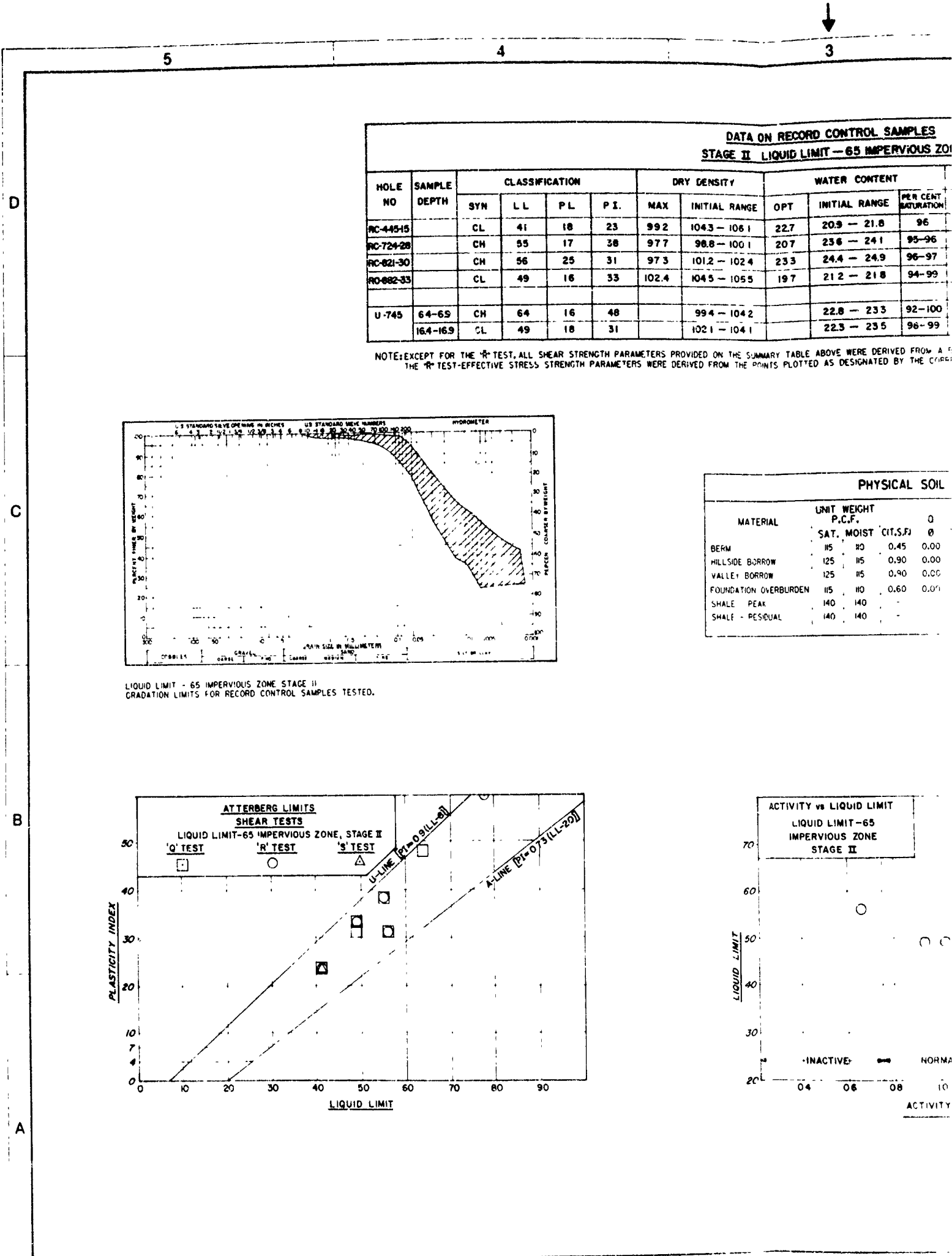
PLATE NO 61







Revisions			
Symbol	Descriptions	Date	Approved
U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	WBE	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by	MAW	SHEAR TEST SUMMARIES LIQUID LIMIT-60 RANDOM ZONE STAGE II	
Checked by	RGW/WBE	Scale	AS SHOWN
Submitted by	PCW	Date	
		Drawn by	
		File No	RBL-3-1462

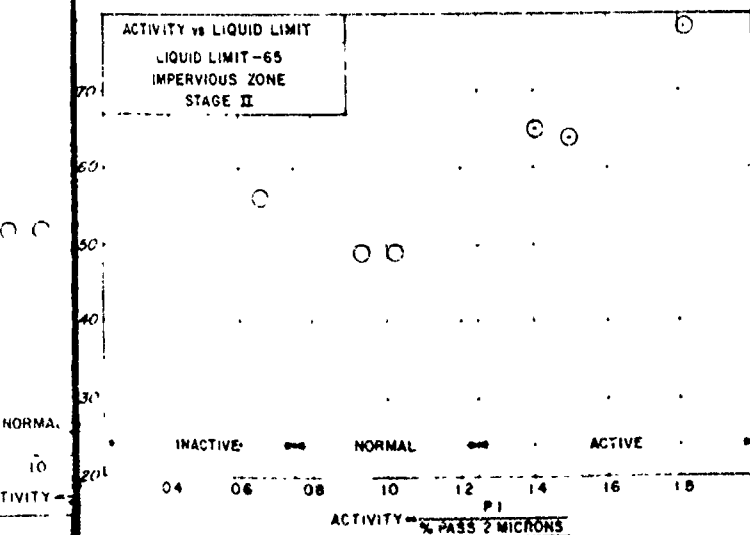


# ES IS ZONE DATA ON RECORD CONTROL SAMPLES II LIQUID LIMIT - 65 IMPERVIOUS ZONE

CENT ATION	RANGE	WATER CONTENT			"Q" TEST		"R" TEST		"R" TEST		"S" TEST DIRECT SHEAR	
		OPT.	INITIAL RANGE	PER CENT SATURATION	TAN Ø	C TSF	TAN Ø	C TSF	TAN Ø	C TSF	TAN Ø	C TSF
-96	0061	22.7	209 - 218	96	0.065	148	0.404	0.13	0.246	0.28	0.505	0
-97	0001	207	236 - 241	95-98	0.099	100	0.521	0.14	0.268	0.53		
-99	0024	233	244 - 249	96-97	0.045	113	0.379	0.17	0.213	0.30		
	0065	197	212 - 218	94-99	0.040	150	0.344	0.21	0.216	0.53		
-100												
-99	0042		22.8 - 23.3	92-100	0.092	148	0.240	0.58	0.194	0.65		
	0041		22.3 - 23.5	96-99	0.070	0.75						

THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT TANGENT TO THE MOHR'S CIRCLES FOR EACH SAMPLE TESTED.  
THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING MOHR'S DIAGRAM.

PHYSICAL SOIL CONSTANTS													
Q	TAN Ø	MATERIAL	UNIT WEIGHT			DESIGN SHEAR STRENGTHS							
			P.C.F.			Q	TAN Ø (C.T.S.F.)	R	TAN Ø (C.T.S.F.)	S	TAN Ø		
0.00	0		SAT. MOIST	(C.T.S.F.)		Ø		Ø		Ø			
0.00	0		15	10	0.45	0.00	0.00	0.10	14.3	0.20	0.00	16.7	0.30
0.00	0		125	15	0.90	0.00	0.00	0.20	14.3	0.20	0.00	19.8	0.36
0.00	0	BORROW	125	15	0.90	0.00	0.00	0.20	14.3	0.20	0.00	26.6	0.50
0.00	0	BORROW	125	15	0.90	0.00	0.00	0.20	14.3	0.20	0.00	26.6	0.50
		ION OVERBURDEN	15	10	0.60	0.00	0.00	0.20	14.3	0.20	0.00	26.6	0.50
		PFAR	140	140	-	-	-	-	-	-	0.00	31.0	0.60
		RESIDUAL	140	140	-	-	-	-	-	-	0.00	16.7	0.30



Symbol	Revisions Descriptions	Date	Approved

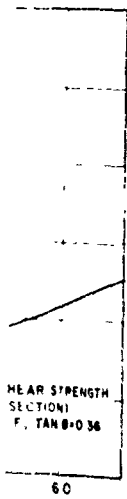
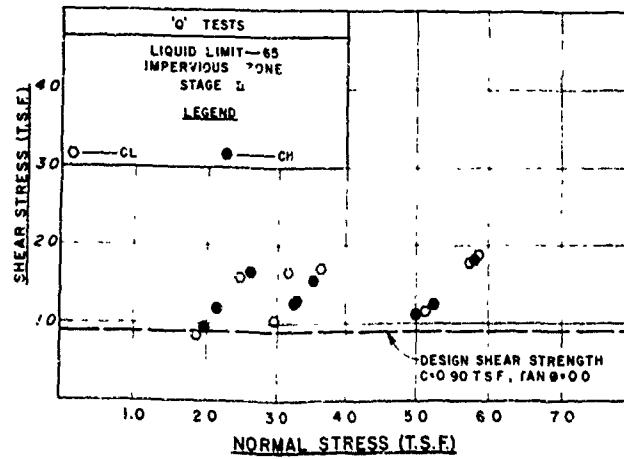
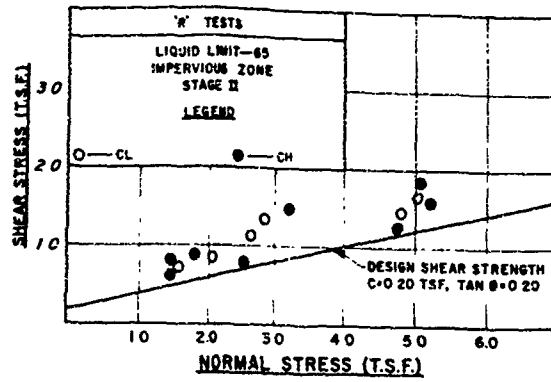
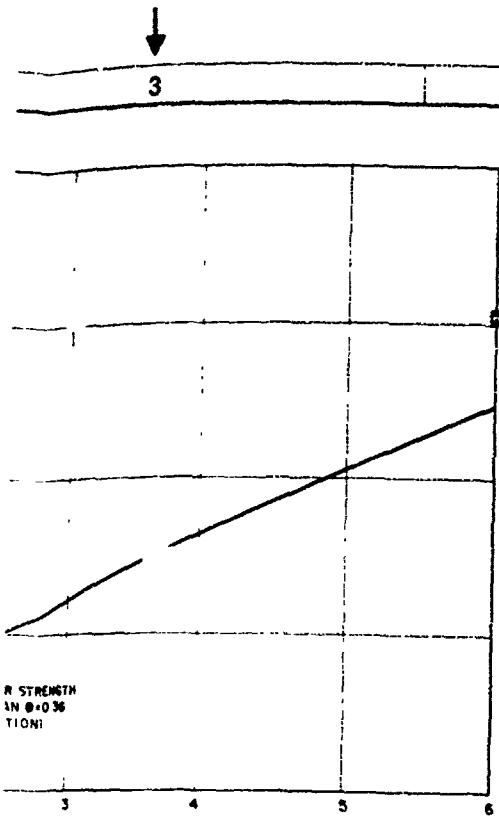
U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
TEST DATA SUMMARIES  
LIQUID LIMIT-65 IMPERVIOUS ZONE  
STAGE II

W. B. E. [Signature]  
U. S. Army Corps  
of Engineers

AS SHOWN

RBL-3-1463



Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: WBE  
Drawn by: JGS  
Checked by: RGD/WBE  
Submitted by: FCW

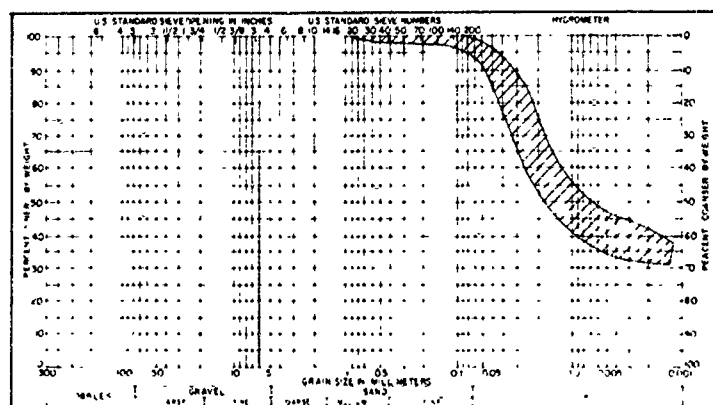
LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
SHEAR TEST SUMMARIES  
LIQUID LIMIT-65 IMPERVIOUS ZONE  
STAGE II

Scale: AS SHOWN  
Date:    
Sheet number:  

FA No. RBL-3-1464

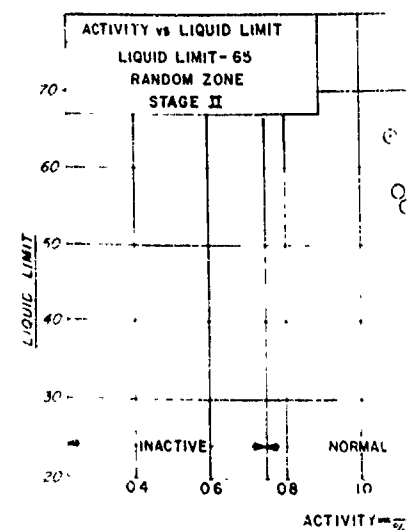
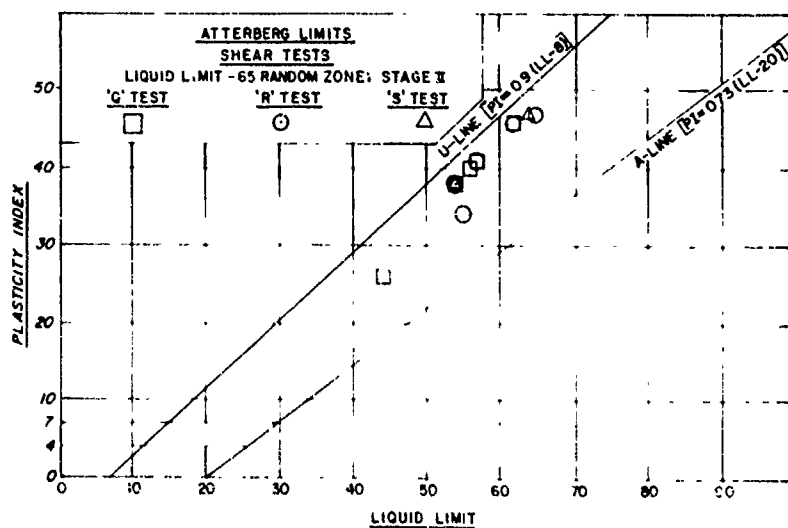
DATA ON RECORD CONTROL SAMPLES STAGE II LIQUID LIMIT-65 RANDOM ZONE										
HOLE NO.	SAMPLE DEPTH	CLASSIFICATION				DRY DENSITY		WATER CONTENT		
		SYN	L L.	PL	P.I.	MAX	INITIAL RANGE	OPT	INITIAL RANGE	PER CENT SATURATION
RC-659-22		CH	57	16	41	99.7	1039 - 1056	20.8	19.5 - 20.5	84-90
RC-592-21		CH	62	16	46	97.5	1010 - 1016	22.6	24.0 - 24.4	99
RC-452-16		CH	54	16	38	100.2	1053 - 1065	21.6	21.2 - 21.9	98
PPE-92-1	58.3									
PPE-95-1	188-193	CH	78	20	58		1032 - 1046		20.7 - 20.8	89-92
	288-293	CH	65	18	47		1041 - 105.0		22.2 - 22.5	100
PPE-93-12	5.4-5.9	CH	55	21	34		1075 - 1094		18.4 - 19.3	93-94

NOTE: EXCEPT FOR THE 'R' TEST, ALL SHEAR STRENGTH PARAMETERS PROVIDED ON THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT TO THE 'R' TEST-EFFECTIVE STRESS STRENGTH PARAMETERS WERE DERIVED FROM THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING SYMBOLS.



LIQUID LIMIT - 65 RANDOM ZONE STAGE II  
GRADATION LIMITS FOR RECORD CONTROL SAMPLES TESTED.

PHYSICAL SOIL CONSTANTS					
MATERIAL	UNIT WEIGHT		DESIGN STRENGTH		
	SAT. MOIST	P.C.F.	φ	TAN φ	CIT. S.F.
BERM	85	80	0.45	0.00	0.00
HILLSIDE BORROW	125	85	0.90	0.00	0.20
VALLEY BORROW	125	85	0.90	0.00	0.20
FOUNDATION OVERBURDEN	85	80	0.60	0.00	0.00
SHALE - PEAK	140	140	-	-	-
SHALE - RESIDUAL	140	140	-	-	-



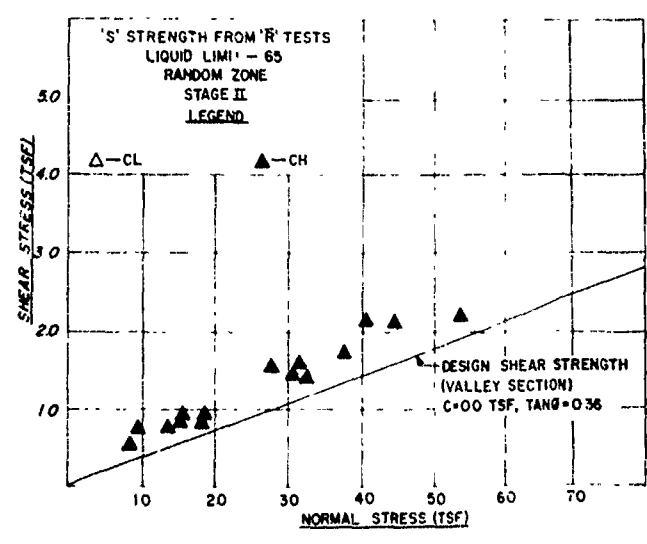
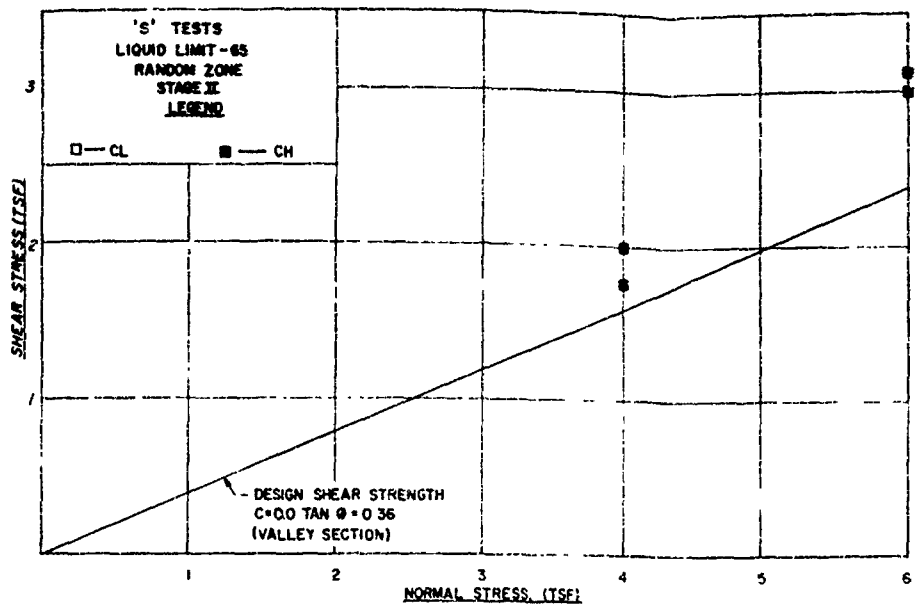
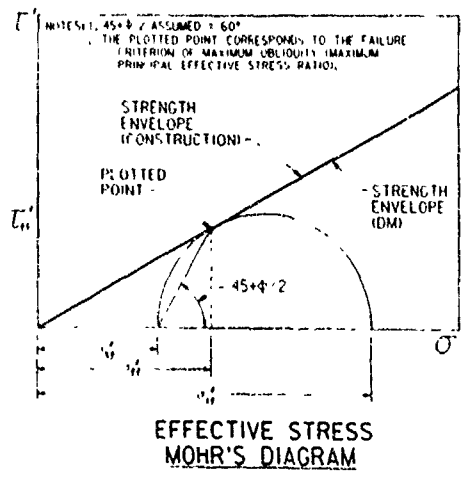
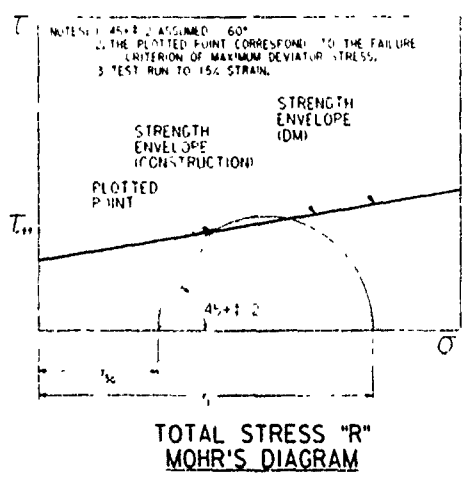
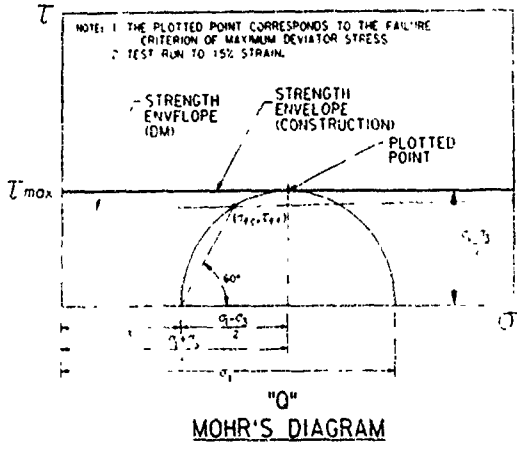


D

C

B

A







5

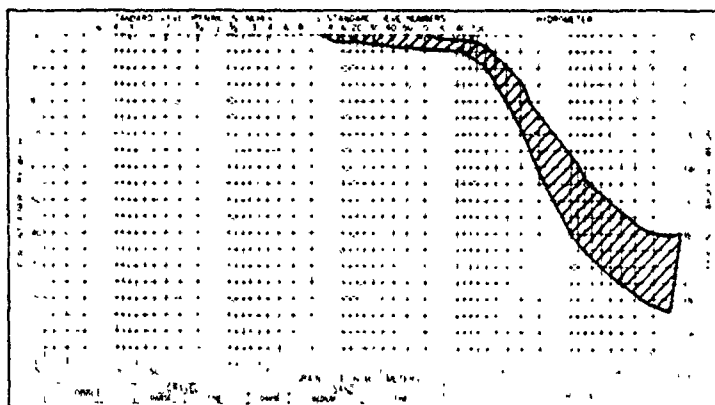
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3

# DATA ON RECORD CONTROL SAMPLES STAGE II LIQUID LIMIT-70 RANDOM ZONE

HOLE NO.	SAMPLE DEPTH	CLASSIFICATION			DRY DENSITY			WATER CONTENT			'O' TEST		'R' TEST		'R' TEST		T <sub>1</sub>
		SYM.	L.L.	P.L.	P.I.	MAX.	INITIAL RANGE	OPT.	INITIAL RANGE	(%) SATURAT.	TAN $\phi$	C T.S.F.	TAN $\phi$	C T.S.F.	TAN $\phi$	C T.S.F.	
PC-855-3		CH	62	19	43	95.6	99.8-103.2	24.8	23.4-25.9	100	0.05	1.80	0.306	0.5	0.249	0.25	
PC-878-32		CH	59	19	40	97.7	99.6-102.2	23.7	22.4-24.4	89-100	0.059	0.98	0.264	0.33	0.187	0.25	
PPE-92-1	1.3-1.8	CH	66	19	47		93.5-101.3		24.3-25.5	98	0	1.75	0.259	0.55	0.374	0.26	
	6.3-6.8	CH	72	20	52		100.9-102.2		22.9-23.2	93-96	0.48	1.38	0.277	0.54	0.394	0.3	
	13.8-15.0						101.2-104.7		21.6-23.6	96							
PPE-95-1	30.8-39.3	CH	78	22	56		98.0-99.5		25.4-25.6	98-99	0.061	1.03	0.306	0.50	0.394	0.4	
	0.0-0.5	CH	57	19	38		99.0-100.2		25.0-25.8	97-98	0.096	0.55	0.466	0.33	0.364	0.3	
	8.3-8.8						98.8-95.4		28.5-31.2	100	0.58	1.05	0.485	0.25	0.222	0.25	
U-749	6.4-6.9	CH	77	19	58		93.4-96.7		28.4-29.2	99-100	0.013	1.48	0.26	0.34	0.149	0.45	
	10.5-11.0	CH	70	16	54		103.7-106.9		20.9-22.3		0	1.50	0.257	0.40	0.154	0.60	
	20.3-20.6	CH	83	19	64		99.8-100.6		23.7-24.2		0	1.10	0.325	0.21	0.173	0.30	

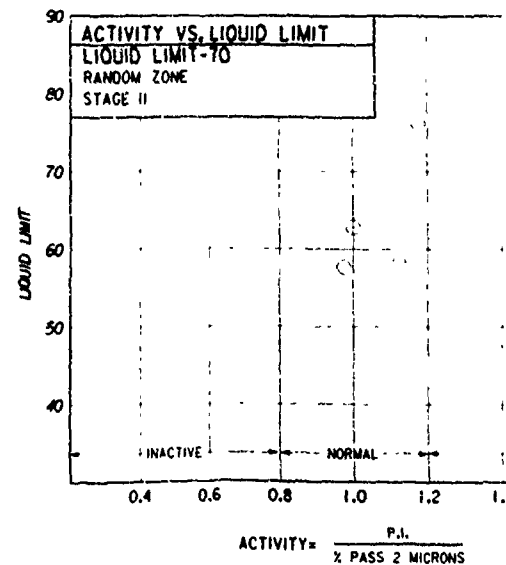
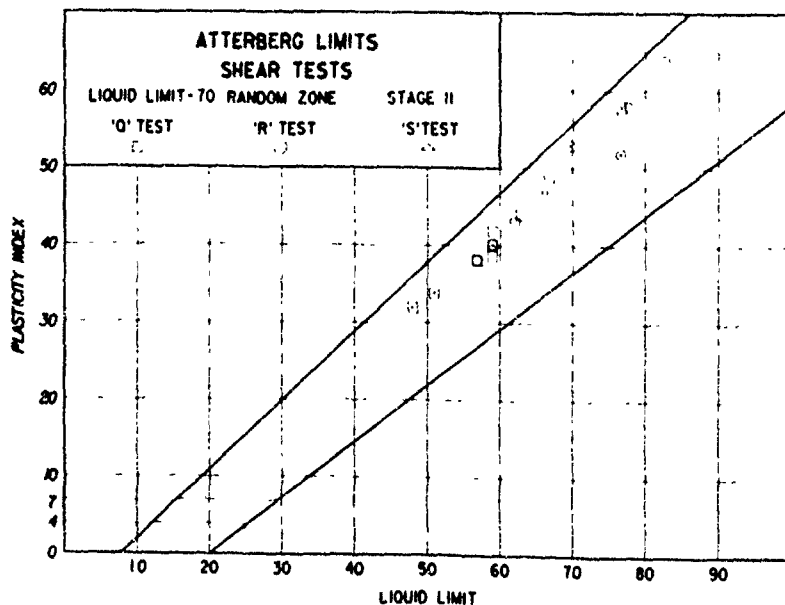
NOTE: EXCEPT FOR THE 'R' TEST, ALL SHEAR STRENGTH PARAMETERS PROVIDED ON THE SUMMARY TABLE ABOVE WERE DERIVED FROM A BEST FIT TANGENT TO THE MOHR'S CIRCLES FOR EACH SAMPLE TEST. THE 'R' TEST-EFFECTIVE STRESS STRENGTH PARAMETERS WERE DERIVED FROM THE POINTS PLOTTED AS DESIGNATED BY THE CORRESPONDING MOHR'S DIAGRAM.



LIQUID LIMIT - 70 RANDOM ZONE STAGE II  
GRADATION LIMITS FOR RECORD CONTROL SAMPLES TESTED.

## PHYSICAL SOIL CONSTANTS

MATERIAL	UNIT WEIGHT		DESIGN SHEAR ST		R
	SAT.	MOIST P.C.F.	TAN $\phi$ (C.T.S.F.)	TAN $\phi$ (C.T.S.F.)	
BERM	95	10	0.45	0.00	0.00
HILLSIDE BORROW	125	15	0.90	0.00	0.20
VALLEY BORROW	125	15	0.90	0.00	0.20
FOUNDATION OVERBURDEN	15	10	0.60	0.00	0.20
SHALE - PEAR	140	140			
SHALE - RESIDUAL	140	140			



VALUE

VALUE ENGINEERING PAYS

3

2

1

"S" TEST DIRECT		"R" TEST		"S" TEST DIRECT SHEAR	
TAN $\theta$	C	TAN $\theta$	C	TAN $\theta$	C
0.306	0.05	0.249	0.25		
0.260	0.03	0.187	0.25		
0.259	0.95	0.374	0.26		
0.277	0.54	0.384	0.3	0.538	0
0.306	0.50	0.394	0.4		
0.466	0.03	0.364	0.03		
0.485	0.25	0.227	0.25		
0.26	0.34	0.499	0.45		
0.257	0.40	0.54	0.60		
0.325	0.21	0.773	0.30		

TO THE MOHR'S CIRCLES FOR EACH SAMPLE TESTED.  
DIAGRAM

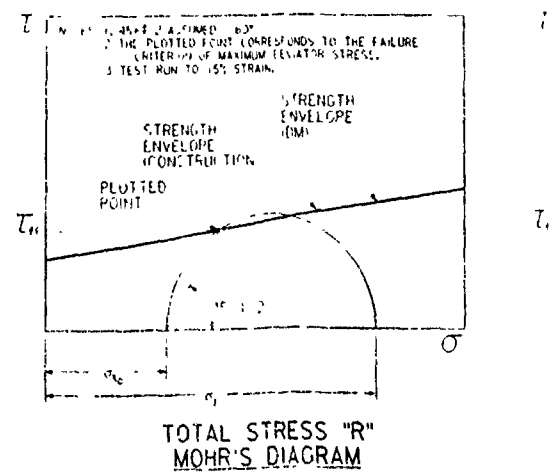
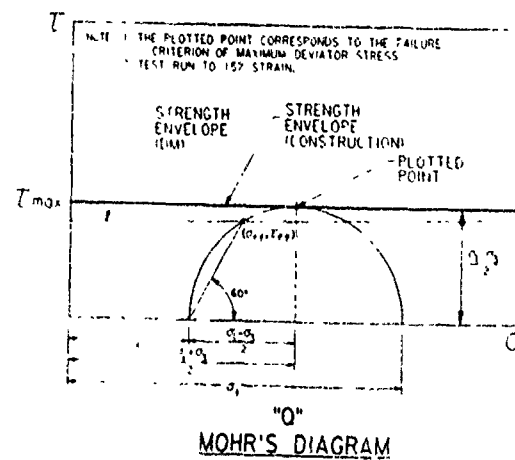
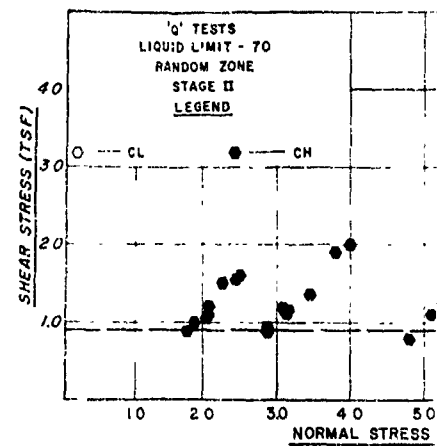
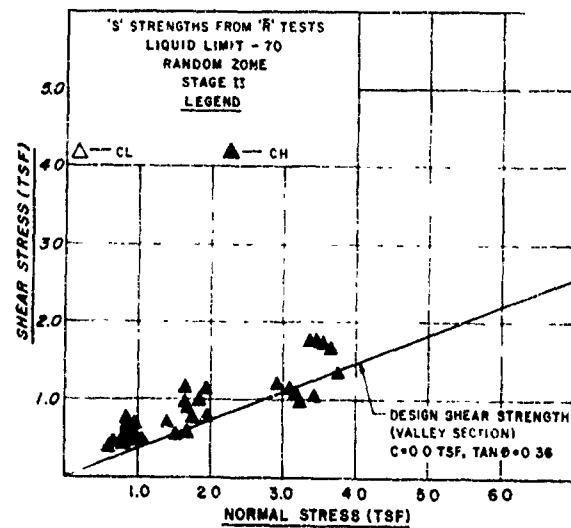
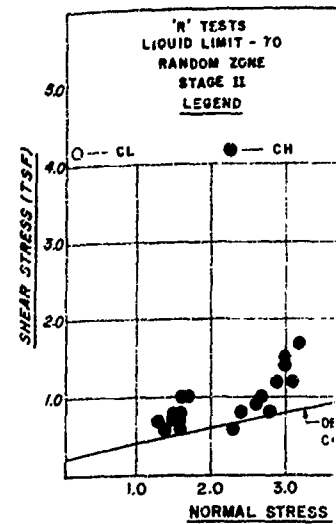
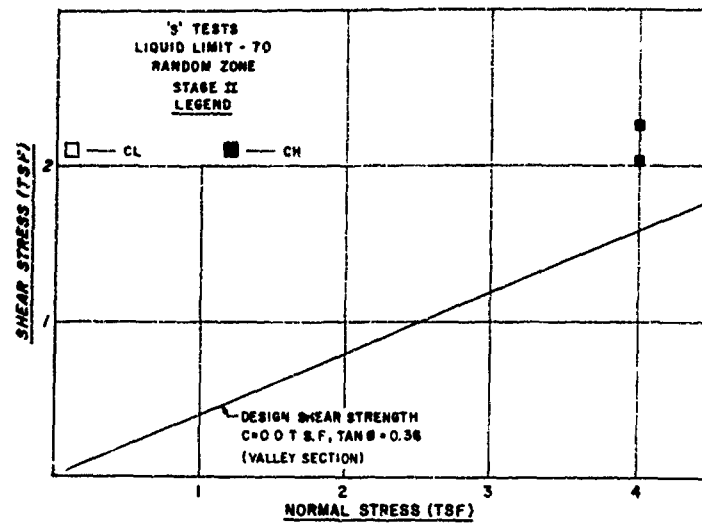
PHYSICAL SOIL CONSTANTS									
WEIGHT		DESIGN SHEAR STRENGTH							
TAN $\theta$	C.F.	0	TAN $\theta$	C.F.	0	TAN $\theta$	C.F.	0	TAN $\theta$
0.3	0.2	MOIST	0.45	0.00	0.00	0.30	0.20	0.00	16.7
0.3	0.2	NO	0.90	0.00	0.00	0.20	0.20	0.00	19.8
0.3	0.2	15	0.90	0.00	0.00	0.20	0.20	0.00	26.6
0.3	0.2	15	0.90	0.00	0.00	0.20	0.20	0.00	26.6
		10	0.60	0.00	0.00	0.20	0.20	0.00	36.0
		140						0.00	36.0
		140						0.00	36.7

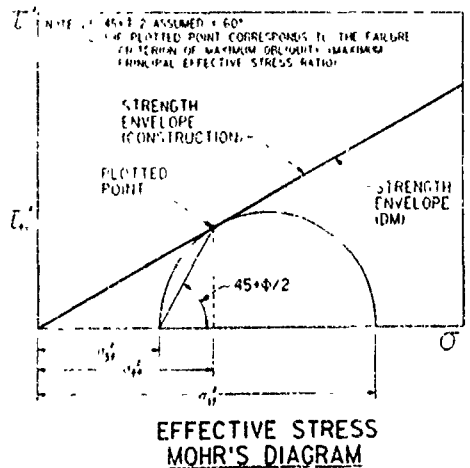
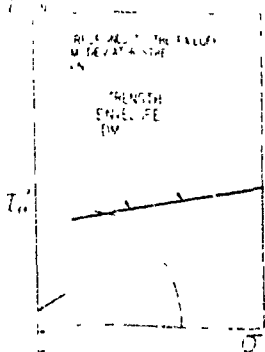
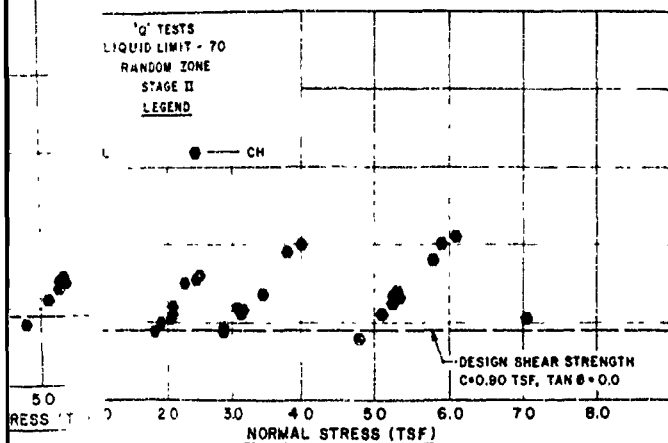
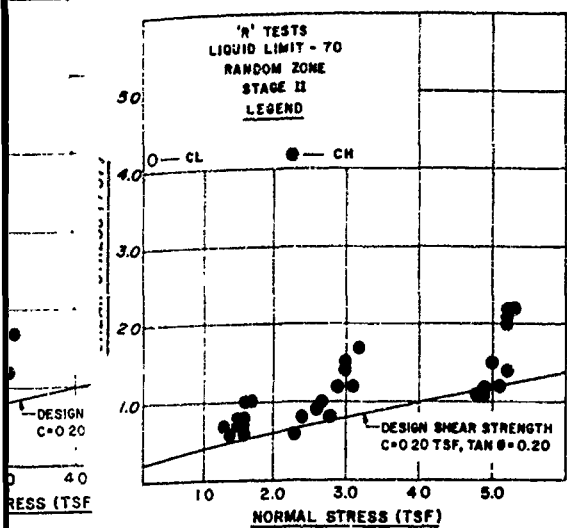
VS. LIQUID LIMIT  
MIT-70  
VE

IVE  
0.6 0.8 1.0 1.2 1.4 1.6 1.8  
NORMAL ACTIVE  
ACTIVITY: P.J.  
% PASS 2 MICRONS

Revisions		Date	Approved
Symbol	Descriptions		
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	R.G.D.	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT TEST DATA SUMMARIES LIQUID LIMIT-70 RANDOM ZONE STAGE II	
Drawn by:	R.A.C.	Scale:	AS SHOWN
Checked by:	A.L.P.	Sheet number:	Plot Scale: 12 11
Submitted by:	R.G.D.	Date:	Design File: 100.62X.VLRPT80
		Dwg. No.:	File No. RBL-3-1467

PLATE NO. 67





Revisions			
Symbol	Descriptions	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: WBE

Drawn by: EGM

Checked by: WBE/RGD

Submitted by: F.C.W.

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
SHEAR TEST SUMMARIES  
LIQUID LIMIT-70 RANDOM ZONE  
STAGE II

Scale: AS SHOWN

Date:  

Sheet Number:  

File No: RBL-3-1468



3

2

TAN B

F.	R TEST		R TEST		S TEST DIRECT SHEAR	
	TAN $\theta$	C T.S.F.	TAN $\theta$	C T.S.F.	TAN $\theta$	C T.S.F.
5	0.404	0.20	0.203	0.35		
5	0.44	0.21	0.35	0.15		
3	0.556	0	0.277	0.08		
	0.384	0.07	0.249	0.23		
	0.364	0.20	0.259	0.325		

7 TO THE MONR'S CIRCLES FOR EACH SAMPLE TESTED.  
5 DIAGRAM.

PHYSICAL SOIL CONSTANTS										
CAR STR		DESIGN SHEAR STRENGTHS								
R	UNIT WEIGHT	O			R			S		
Ø	P.C.F.	Ø	TAN Ø	CIT.S.F.	R	TAN Ø	CIT.S.F.	S	TAN Ø	
	SAT. MOIST	(Ø.T.S.F.)								
U.3	15	10	0.45	0.00	0.00	0.00	0.00	16.7	0.30	
P.3	15	15	0.90	0.00	0.00	0.20	0.00	19.8	0.36	
P.4	25	15	0.90	0.00	0.00	0.20	0.00	26.6	0.50	
		10	0.60	0.00	0.00	0.20	0.00	26.6	0.50	
	40	140					0.00	31.0	0.60	
	41	142					0.00	16.7	0.30	

10 LIMIT  
SPECIFIED

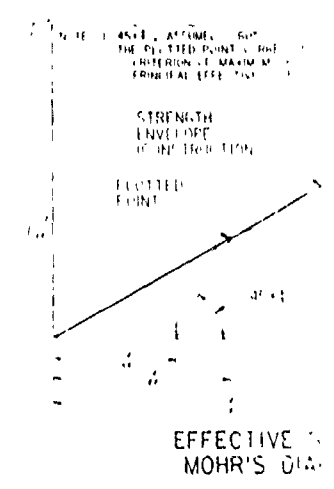
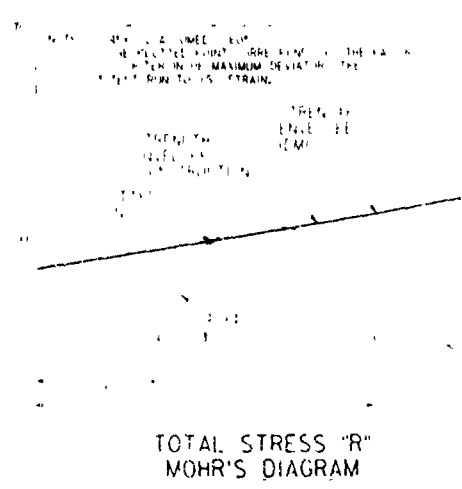
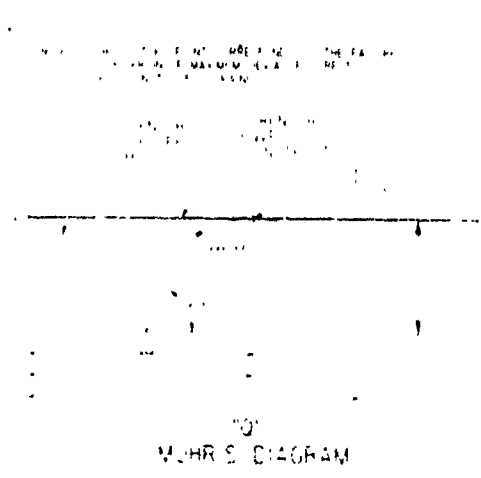
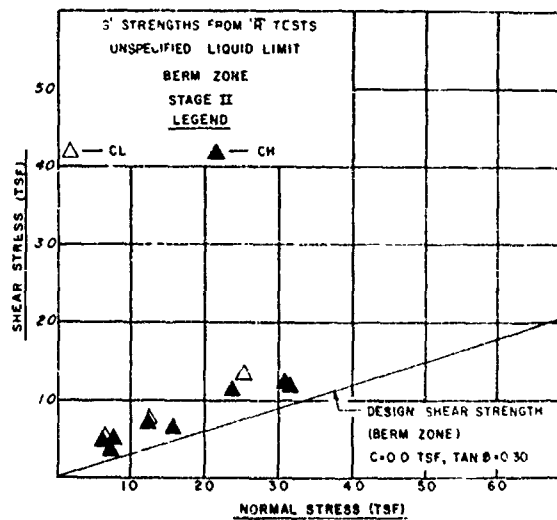
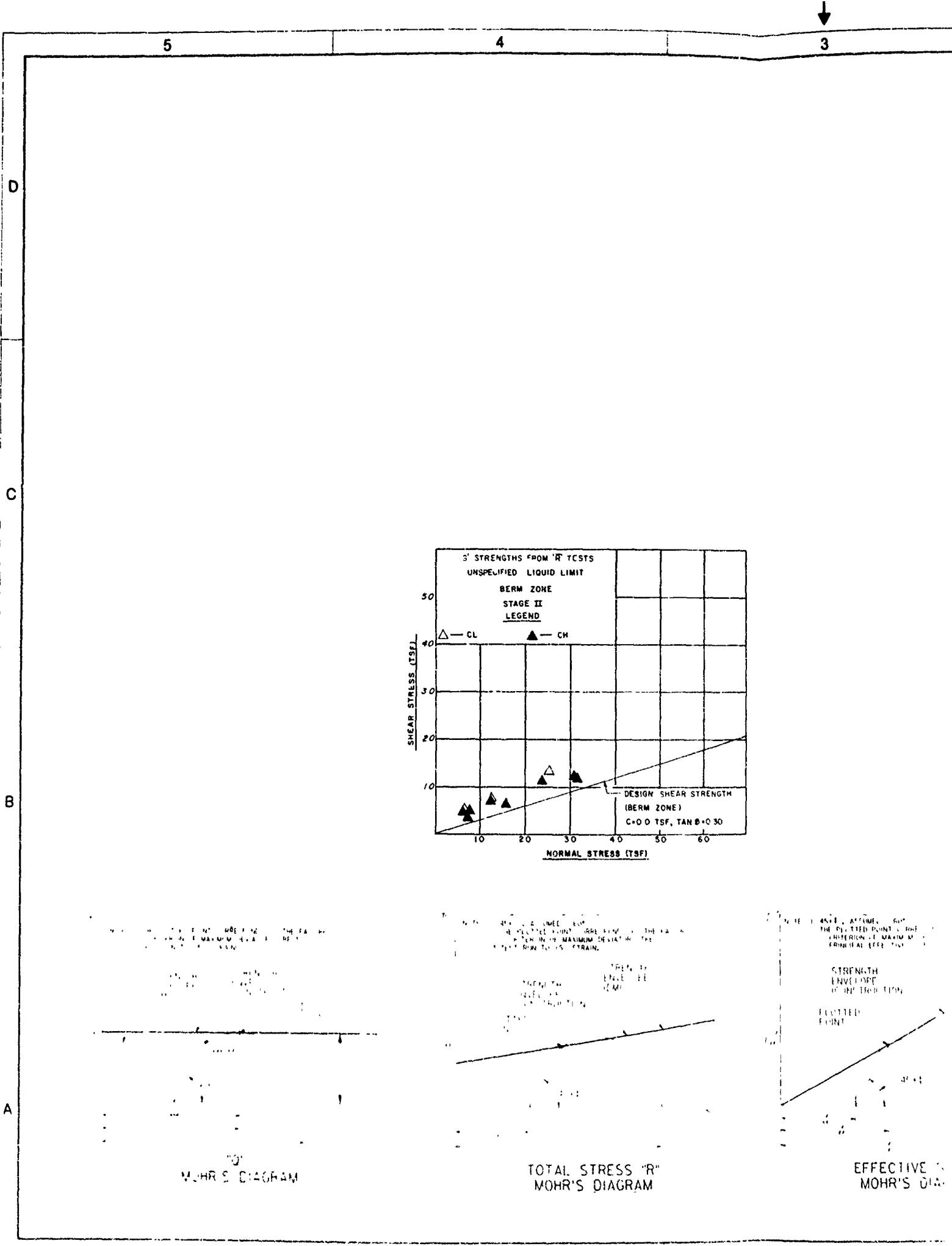
1. The first step is to identify the problem or question that needs to be answered. This involves understanding the context and the specific requirements of the task.

TEST DATA SUMMARIES  
UNSPECIFIED LIQUID LIMIT BERM ZONE  
STAGE 11

RA 1469

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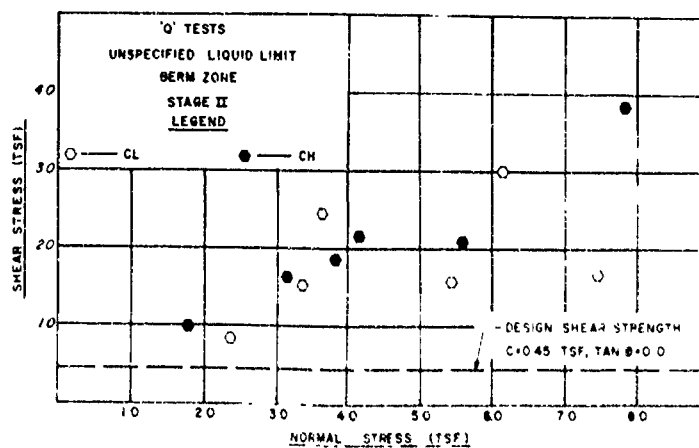
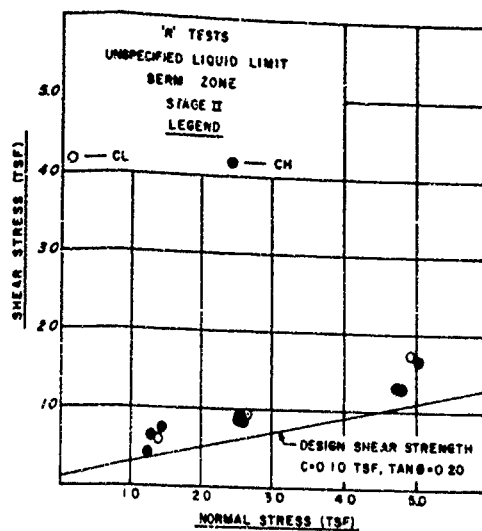




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EFFECTIVE STRESS  
MOHR'S DIAGRAM

DATE	10/1/61	BY	W
PROJECT	UNSPECIFIED LIQUID LIMIT BERM ZONE STAGE II		
DESCRIPTION	SHEAR TEST SUMMARIES		
LOCATION	UNSPECIFIED LIQUID LIMIT BERM ZONE STAGE II		
TEST TYPE	SHEAR TEST		
TEST NUMBER	RBL-3-1470		

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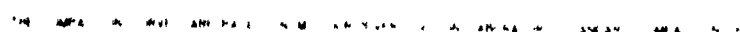
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PLATE NO. 10

## DENSITY:

THE RE ONLY CONTROL SAMPLES WERE A "KID" AND "MAY" SA REEDS WERE  
IN THE UPPER PORTION OF THE "TONE" TRENCH



DENSITY

NE DA

DENSITY AND MOISTURE CONTENT											
COMPACTION TEST				DENSITY AND MOISTURE CONTENT							
M.R.D. LAB				SAND CONE DATA				* UNDISTURBED DATA			
WATER CONTENT (%)	OPT. MOIST. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. (%)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+/- 20PT)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+/- 20PT)
23.7	97.2	23.5	96.6	99.4	26.8	+3.3		96.6	99.4	27.4	+3.9
19.3	105.2	16.5	105.9	100.7	20.1	+4.6		107.0	104.7	20.3	+3.8
19.8	105.0	17.8	107.3	102.2	19.0	+1.2		107.7	102.6	19.2	+1.4
23.0	97.4	26.2	100.4	103.1	24.8	+3.6		99.6	102.3	23.6	+2.4
9.7	103.0	19.0	109.2	106.0	19.9	+0.9		107.7	104.6	19.2	+0.2
9.0	102.2	18.7	107.9	105.6	20.5	+1.8		106.5	104.2	20.6	+1.9
10.3	103.8	18.8	104.3	100.5	21.9	+3.1		100.1	97.4	24.3	+5.5
20.5	99.7	21.3	100.5	100.8	23.8	+2.5		96.4	96.7	27.2	+5.9
10.5	105.3	18.7	102.3	97.2	20.1	+2.4		104.4	99.1	21.9	+3.2
10.6	106.0	20.3	98.9	90.5	22.8	+2.5		99.7	98.1	24.7	+4.4
10.6	105.6	24.3	98.0	102.5	23.5	+0.8		99.9	104.5	23.0	+1.1
9.7	97.7	22.2	104.7	107.2	19.7	+2.5		106.9	109.4	20.4	+1.8

RIVER

LIMIT-55  
IMPERVIOUS ZONE  
STAGE I

MISSOURI RIVER DIVISION LABORATORY DATA

DEPTH	WATER CONTENT (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	SATURATION %	LL	PL	PI	SPECIFIC GRAVITY
1	22.1	105.0	100				
2	20.3	97.6	85				
3	17.0	107.0	98				
4	10.6	100.6	96				
5	10.6	101.6	94	42	20	22	
6	10.3	104.7	97				
7	10.7	106.9	95				
8	10.2	101.2	95				
9	10.6	102.8	99				2.64
10	8.6	106.7	92				2.62
11	10.4	107.0	89				2.66
12	8.6	109.8	98				2.65
13	10.7	101.0	96				2.65
14	10.1	109.4	97				2.62
15	22.7	101.4	97				2.63
16	18.9	104.8	87				
17	19.2	104.8	88	42	17	25	
18	20.0	103.6	89				
19	19.0	105.7	90				
20	18.8	107.4	92	42	17	25	
21	20.5	103.0	91	43	18	25	2.60
22	20.6	96.9	79				
23	9.9	104.3	93				
24	19.9	104.2	93				
25	20.5	99.5	85				
26	20.8	100.0	89				
27	20.8	100.3	90	49	16	33	2.60

NOTE: ALL TESTS WERE RUN AT A SPECIFIC GRAVITY OF 2.65

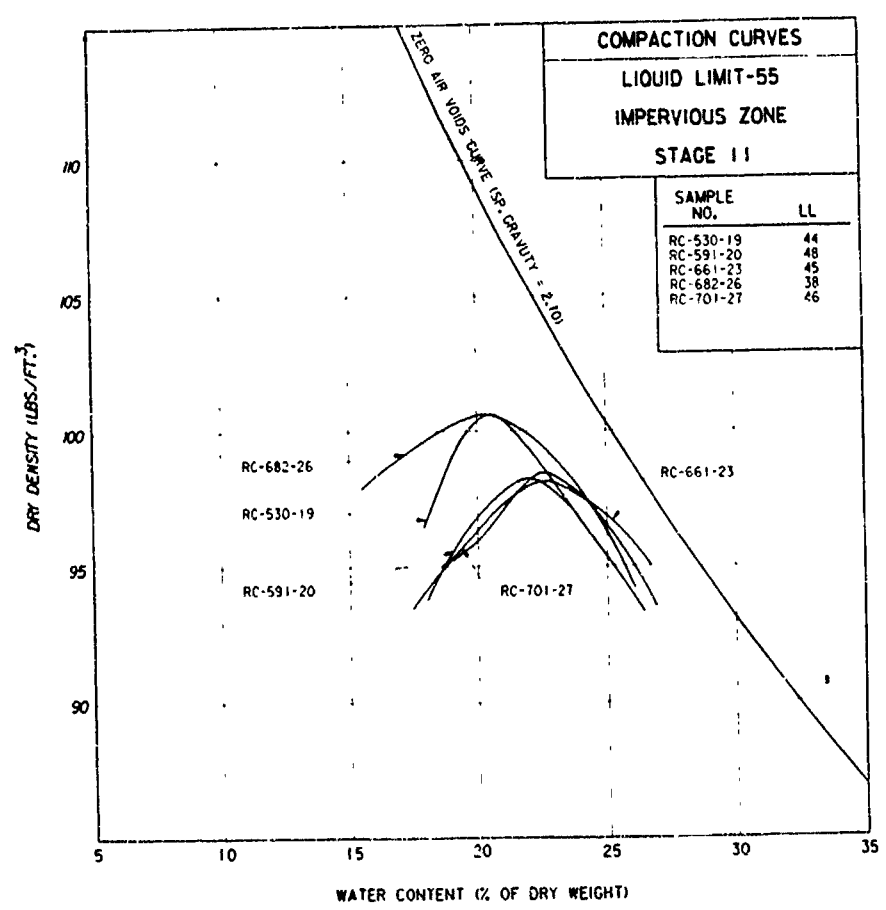
NOTE: ALL TESTS WERE RUN ON A 36 INCH DIAMETER BUCKET AUGER HOLE.

Revisions	Date	Approved
Descriptions		
<p>U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p>		
Designed by	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT LIQUID LIMIT-55 IMPERVIOUS ZONE STAGE I DENSITY AND COMPACTION TEST SUMMARIES</p>	
Drawn by	<p>R.G.D.</p>	
Checked by	<p>R.D.T.</p>	
Submitted by	<p>A.L.P.</p>	
AS SHOWN	Sheet	Page
	Number	Design
		File
		No.
		RBL-3-1471

LIQUID LIMIT-55 IMPERVIOUS ZONE, STAGE II

RECORD CONTROL SAMPLE				SOIL CLASSIFICATION DATA									STANDARD COMPACTION TEST					
				FIELD			M.R.D. LABORATORY						FIELD		M.R.D. LAB		SAND CONE	
NUMBER	STATION	RANGE	ELEV.	LL	PI	CLASS	SHEAR TEST SPECIMEN UNDISTURBED SAMPLE			COMPACTION TEST SPECIMEN SACK SAMPLE			MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	COMPACTION (%)
RC-530-19	94+25	0+50u/s	815.0	43	18	CL	48	32	CL	44	26	CL	104.0	22.0	100.6	20.6	102.9	102.3
RC-591-20	95+50	0+65u/s	828.0	49	27	CL	55	40	Ch	48	31	CL	106.6	22.5	98.3	22.1	97.5	99.2
RC-661-23	95+50	0+30u/s	857.0	51	26	CH	46	30	CL	45	28	CL	100.5	22.2	98.1	22.7	99.2	101.1
RC-682-26	93+00	0+25u/s	862.0	45	26	CL	44	28	CL	38	23	CL	100.5	21.5	100.5	19.8	103.5	103.7
RC-701-27	99+30	1+85u/s	865.0	44	22	CL	51	33	CH	46	28	CL	98.0	22.5	98.5	22.5	99.4	100.3

\* UNDISTURBED DATA OBTAINED BY AVERAGING MOISTURE AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS.  
 NOTE: ALL DRY DENSITIES AND WATER CONTENTS ARE COMPARED TO M.R.D. DATA TO OBTAIN PERCENT COMPACTION AND WATER CONTENT PERCENT ± OPTIMUM.



THE COMPACTION CURVES ARE BASED ON MISSOURI RIVER DIVISION LABORATORY STANDARD COMPACTION TESTS.

DENSITY TEST					
LIQUID LIMIT-55 IMPERVIOUS ZONE STAGE II		K.C.D. LAB SAND CONE			
SAMPLE	DEPTH	W (%)	DRY DENSITY (LBS./FT.³)	L.L.	P.L.
RC-530-19		20.9	102.9	43	25
RC-591-20		24.5	97.5	49	22
RC-661-23		21.2	98.1	48	21
RC-682-26		17.7	98.6	45	19
RC-701-27		21.0	95.9	44	22


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STANDARD COMPACTION TEST DENSITY AND MOISTURE CONTENT												
FIELD			M.R.D. LAB				SAND CONE DATA			*UNDISTURBED DATA		
COM- ACTION (%)	DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	COM- PACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-%OPT)	DRY DENSITY (P.C.F.)	COM- PACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-%OPT)
102.3	4.0	22.0	100.6	20.6	102.9	102.3	20.3	+0.5	100.0	99.4	20.7	+0.1
99.2	4.6	22.5	98.3	22.1	97.5	99.2	24.5	+2.4	101.0	102.8	23.2	+1.1
101	5.5	22.2	98.1	22.7	99.2	101	23.4	-0.7	99.6	101.5	23.8	+1.1
100.9	5.5	21.5	100.5	19.8	103.5	103.0	19.6	-0.2	101.5	101.0	18.9	-0.9
1.0	22.5	98.5	22.5	99.4	100.9	23.5	+1.0	98.1	99.6	23.4	+0.9	

DENSITY TEST SUMMARY											
MIT-55 JS ZONE		K.C.D. LAB					M.R.D. LAB				
		SAND CONE					RE ORD. CRITERIA UNDISTURBED				
P.L.	DEPTH	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.
25		20.9	102.9	43	25	18	20.7	100.0	48	16	37
22		24.5	97.5	49	22	27	23.2	101.0	55	15	40
21		21.2	98.1	48	21	27	23.8	99.6	46	16	30
19		17.7	98.6	45	19	26	18.9	101.5	44	16	28
22		21.0	95.9	44	22	22	23.4	98.1	51	18	31

Revisions		Date	Approved
Signature	Descriptions		
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	R.C.D.	 LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT LIQUID LIMIT-55 IMPERVIOUS ZONE STAGE II DENSITY AND COMPACTION TEST SUMMARIES	
Drawn by	R.D.T.		
Checked by	A.L.P.		
Submitted by	R.C.D.		
Scale	AS SHOWN	Sheet numbers	Plot Scale
Date			Design File
Dwg No			File No
			RBL-3-1472

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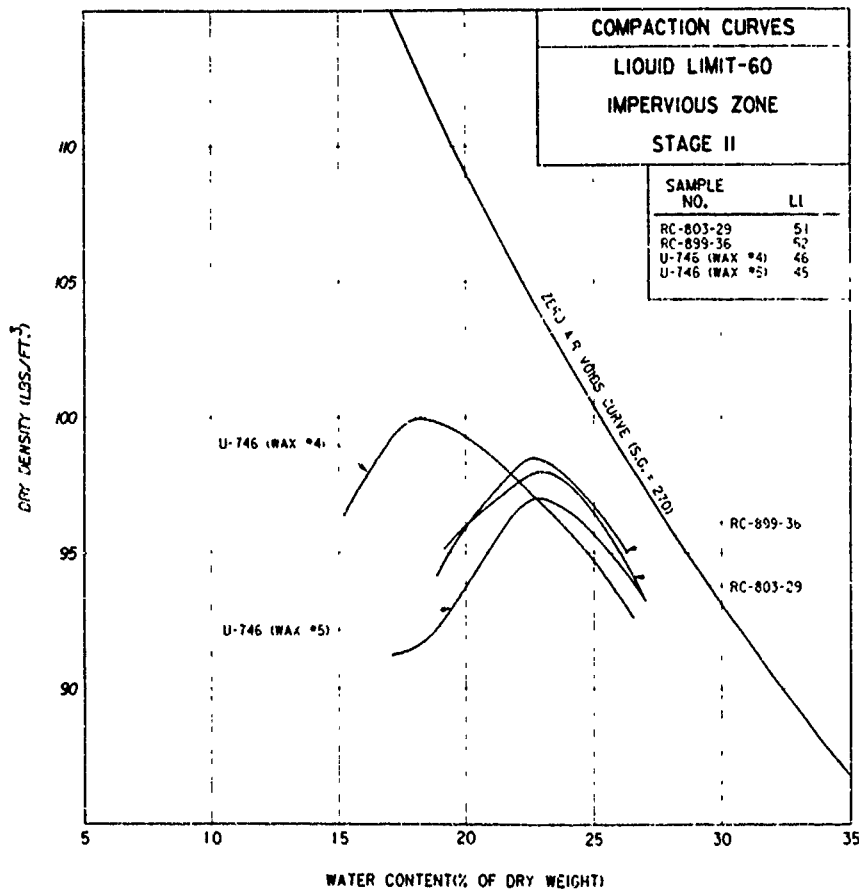
LIQUID LIMIT-60 IMPERVIOUS ZONE, STAGE II																		
RECORD CONTROL SAMPLE				SOIL CLASSIFICATION DATA									STANDARD COMPACTION TEST					
				FIELD			M.R.D. LABORATORY						FIELD		M.R.D. LAB		SAND	
NUMBER	STATION	RANGE	ELEV.	LL	PI	CLASS	SHEAR TEST SPECIMEN UNDISTURBED SAMPLE			COMPACTION TEST SPECIMEN SACK SAMPLE			MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)		DRY DENSITY (P.C.F.)
							LL	PI	CLASS	LL	PI	CLASS					LL	
RC-803-29	99+00	0+90u/s	890.0	54	31	CH	53	29	CH	51	29	CH	98.9	23.3	98.0	23.2	96.1	
RC-899-36	98+00	1+50u/s	898.0	64	43	CH				52	34	CH	99.3	24.2	98.5	22.7	104.5	
U-746*4	98+40	1+70u/s	828.0				43	25	CL	46	29	CL			100.0**	18.3		
U-746*5	98+40	1+70u/s	826.3				43	25	CL	45	26	CL			97.2**	22.2		

\*UNDISTURBED DATA OBTAINED BY AVERAGING MOISTURE AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS.

NOTE: ALL DRY DENSITIES AND WATER CONTENTS ARE COMPARED TO M.R.D. DATA TO OBTAIN PERCENT COMPACTION AND WATER CONTENT PERCENT ± OPTIMUM.

\*\*HARVARD MINIATURE COMPACTION TEST

C



THE COMPACTION CURVES ARE BASED ON MISSOURI RIVER DIVISION LABORATORY STANDARD COMPACTION TESTS.

DENSITY				
LIQUID LIMIT-60 IMPERVIOUS ZONE STAGE II		K.C.D. LAB		
SAMPLE	DEPTH	W (%)	DRY DENSITY (LBS./FT.³)	LI
RC-803-29		25.6	96.1	
U-746	70 to 71'			
J-16	72.3-72.8			

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VALUE ENGINEERING PAYS

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SAND

STANDARD COMPACTION TEST					DENSITY AND MOISTURE CONTENT							
FIELD		M.R.D. LAB			SAND CONE DATA				* UNDISTURBED DATA			
W	W	W	W	W	W	W	W	W	W	W	W	W
(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
98.9	23.3	98.0	23.2	96.1	98.1	25.6	+2.4	106.0	103.1	22.7	0.5	
99.5	24.2	98.5	22.7	104.5	106.1	23.1	+0.4					
		100.0	18.3					105.6	105.6	21.0	+2.8	
		97.2	22.2					99.0	101.9	25.6	+3.4	

SITY

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## DENSITY TEST SUMMARY

LAB		K.C.D. LAB					M.R.D. LAB				
DID LIMIT-60		K.C.D. LAB					M.R.D. LAB				
SERVING'S ZONE		K.C.D. LAB					M.R.D. LAB				
UE II		K.C.D. LAB					M.R.D. LAB				
W	W	W	W	W	W	W	W	W	W	W	W
(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	(%)
25.6	96	51	11	3		23.3	97.2	22.2			

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U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURILITTLE BLUE RIVER, MISSOURI  
LONGVIEW DAM  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
LIQUID LIMIT-60 IMPERVIOUS ZONE  
STAGE II DENSITY AND  
COMPACTION TEST SUMMARIESR.G.D.  
1st Army Corps  
Civil Engineers

R.A.C.

A.L.P.  
AS SHOWN

R.G.D.

RBL-3-1473

PLATE NO. 73

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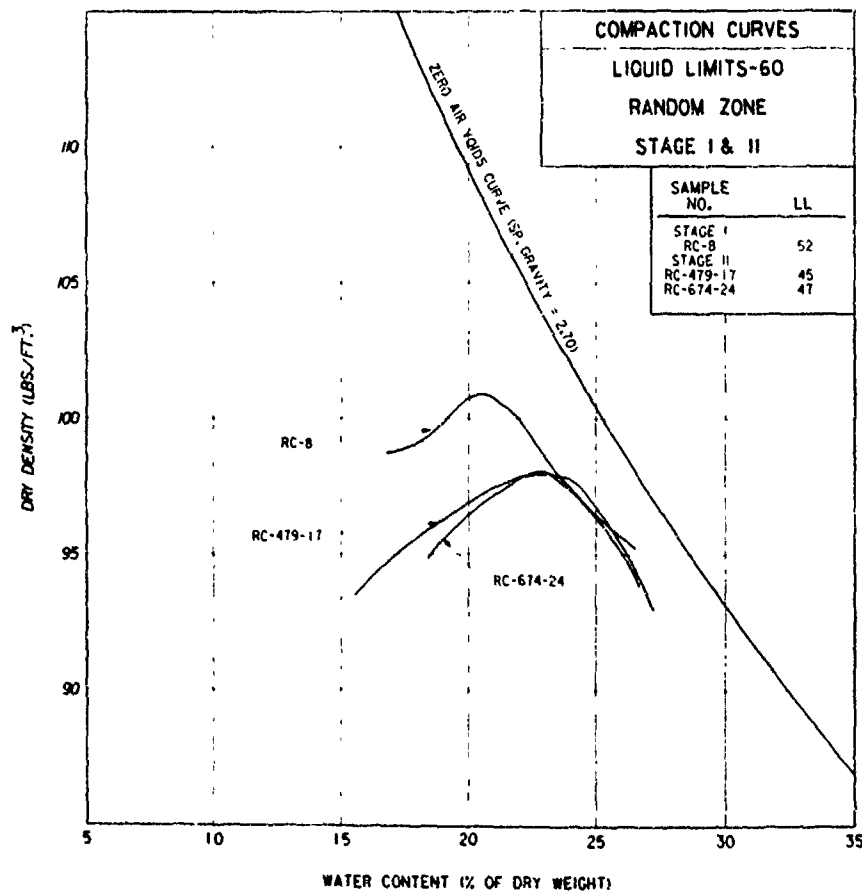
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LIQUID LIMIT-60 RANDOM ZONE, STAGE I																	
RECORD CONTROL SAMPLE				SOIL CLASSIFICATION DATA									STANDARD COMPACTION TEST				
				FIELD			M.R.D. LABORATORY						FIELD		M.R.D. LAB		SAN
NUMBER	STATION	RANGE	ELEV.	LL	PI	CLASS	SHEAR TEST SPECIMEN <small>UNDURABLE SAMPLE</small>			COMPACTION TEST SPECIMEN <small>4 x 1.5 IN.</small>			MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	
							LL	PI	CLASS	LL	PI	CLASS					LL
RC-8	92+00	1+30u/s	821.7	50		CH	51	36	CH	52	35	CH	105.2	20.3	100.8	20.4	105.6
LIQUID LIMIT-60 RANDOM ZONE, STAGE II																	
RC-479-17	93+00	1+80u/s	818.0	55	32	CH	52	34	CH	45	29	CL	98.6	22.5	97.8	22.7	99.5
RC-674-24	94+75	1+10u/s	861.5	55	33	CH	56	41	CH	47	34	CL	100.8	21.0	96.0	22.8	102.4

\* UNDISTURBED DATA OBTAINED BY AVERAGING MOISTURE AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS.

NOTE: ALL DRY DENSITIES AND WATER CONTENTS ARE COMPARED TO M.R.D. DATA TO OBTAIN PERCENT COMPACTION AND WATER CONTENT PERCENT ± OPTIMUM.



LIQUID LIMIT-60 RANDOM ZONE STAGE II			
SAMPLE	ELEV.	W (%)	DRY DENSITY (LBS./FT.³)
RC-479-17	818.0	24.0	98.4
RC-674-24	861.5	20.6	97.5
*PPE-94-1	852.5		



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SAMPLE NO.	STANDARD COMPACTION TEST				DENSITY AND MOISTURE CONTENT							
	FIELD		M.R.D. LAB		SAND CONE DATA				*UNDISTURBED DATA			
	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+/-2.0PT)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+/-2.0PT)
105.6	105.2	20.3	100.8	20.4	105.6	104.8	22.5	+2.0	100.6	99.8	24.3	+3.9
99.5	98.6	22.5	97.8	22.7	99.5	101.7	23.8	+1.1	104.4	106.8	22.3	-0.4
102.4	100.8	21.0	98.0	22.8	102.4	104.5	22.1	-0.4	98.6	100.6	24.3	+1.5

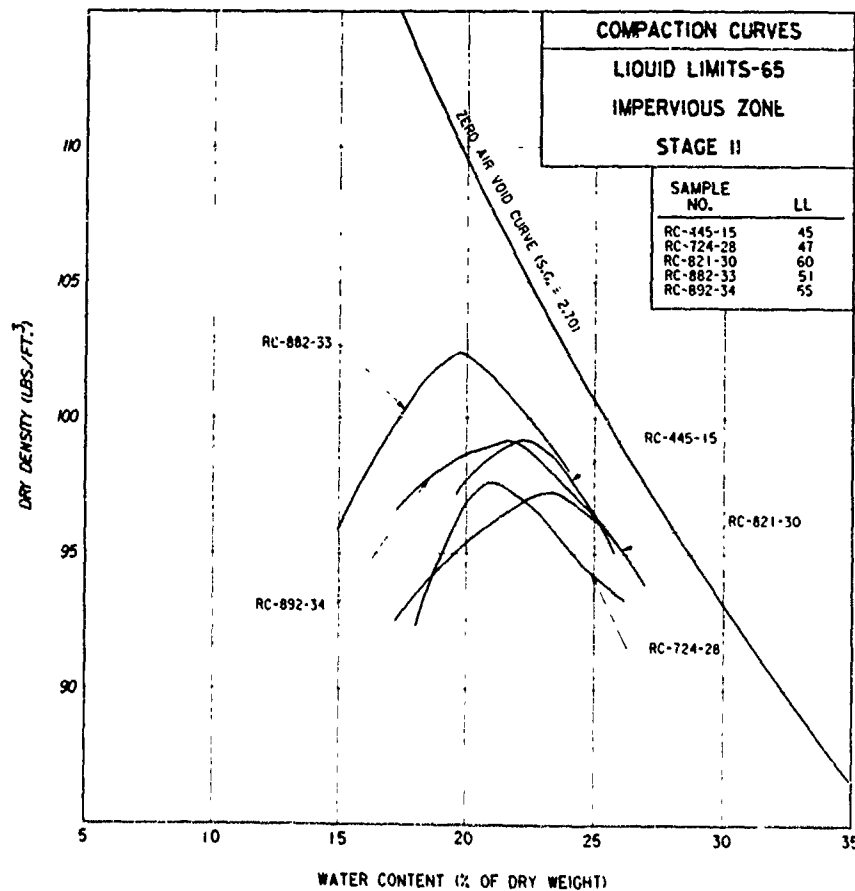
DENSITY TEST SUMMARY											
LIQUID LIMIT-60 RANDOM ZONE STAGE II		K.C.D. LAB					M.R.D. LAB				
SAMPLE	ELEV.	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.
RC-470-17	818.0	24.0	98.4	53	22	31	22.3	104.4	52	18	34
RC-614-24	861.5	20.6	97.5	55	22	33	24.3	98.6	56	15	41
*PPE-94-1	852.5						23.7	99.4	53	17	36

Revisions			
Symbol	Descriptions	Date	Approved
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	R.G.D.	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT LIQUID LIMIT-60 RANDOM ZONE STAGES I AND II DENSITY AND COMPACTION TEST SUMMARIES	
Drawn by:	R.A.C.		
Checked by:	A.L.P.		
Submitted by:	R.G.D.		
Scale:	AS SHOWN	Sheet Number:	Plot Scale:
Date:		Design File:	
Dwg No.:		File No.:	RBL-3-1474

LIQUID LIMIT-65 IMPERVIOUS ZONE; STAGE II																		
RECORD CONTROL SAMPLE				SOIL CLASSIFICATION DATA									STANDARD COMPACTION TEST					
				FIELD			M.R.D. LABORATORY						FIELD		M.R.D. LAB		SANC	
NUMBER	STATION	RANGE	ELEV.	LL	PI	CLASS	SHEAR TEST SPECIMEN UNDISTURBED SAMPLE			COMPACTION TEST SPECIMEN SAG SAMPLE			MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	C/PAC
							LL	PI	CLASS	LL	PI	CLASS						
RC-445-15	100+10	2+30u/s	896.0	49	27	CL	41	23	CL	45	28	CL	99.4	23.0	99.2	22.7	102.4	10
RC-724-28	90+20	0+05d/s	874.0	55	30	CH	55	38	CH	47	32	CL	97.5	24.2	97.7	20.7	97.4	5
RC-821-30	93+20	0+10d/s	900.0	56	34	CH	56	31	CH	60	43	CH	99.4	24.3	97.3	23.3	98.2	10
RC-882-33	106+15	0+08d/s	921.0	43	27	CL	49	33	CL	51	37	CH	102.2	23.0	102.4	19.7	98.1	5
RC-892-34	99+75	0+10d/s	923.5	53	34	CH				55	40	CH	103.0	22.3	99.2	21.5	103.8	10

\* UNDISTURBED DATA OBTAINED BY AVERAGING MOISTURE AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS.

NOTE: ALL DRY DENSITIES AND WATER CONTENTS ARE COMPARED TO M.R.D. DATA TO OBTAIN PERCENT COMPACTION AND WATER CONTENT PERCENT ± OPTIMUM.



THE COMPACTION CURVES ARE BASED ON MISSOURI RIVER DIVISION LABORATORY STANDARD COMPACTION TESTS.

DENSITY				
LIQUID LIMIT-65 IMPERVIOUS ZONE STAGE II		K.C.D. LAB		
SAMPLE	DEPTH	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.
RC-445-15		22.6	102.4	45
RC-724-28		23.9	96.4	55
RC-821-30		24.7	98.2	56

STANDARD COMPACTION TEST				DENSITY AND MOISTURE CONTENT							
FIELD		M.R.D. LAB		SAND CONE DATA				* UNDISTURBED DATA			
MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	COM-PACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-% OPT)	DRY DENSITY (P.C.F.)	COM-PACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-% OPT)
99.4	23.0	99.2	22.7	102.4	103.2	22.6	-0.1	105.9	106.8	21.0	-1.7
97.5	24.2	97.7	20.7	97.4	99.7	25.2	+4.5	98.2	100.5	23.9	+3.2
99.4	24.3	97.3	21.3	98.2	100.9	24.7	+1.4	100.4	103.2	24.8	+1.5
102.2	23.0	102.4	19.7	98.1	95.8	22.4	+2.7	104.1	101.7	21.2	+1.5
103.0	22.3	99.2	21.5	103.8	104.6	20.3	-1.2				

DENSITY TEST SUMMARY											
LIQUID LIMIT-65 IMPERVIOUS ZONE STAGE II			K.C.D. LAB				M.R.D. LAB				
SAMPLE	DEPTH	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.
RC-445-15		22.6	102.4	49	27	22	21.0	105.9	41	18	23
RC-724-26		23.9	96.4	55	25	30	23.9	98.2	55	17	38
RC-821-30		24.7	98.2	56	22	34	24.8	100.4	56	25	31

Revisions	Date	Approved
Descriptions		
<p>U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI</p>		
Designed by	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT LIQUID LIMIT-65 IMPERVIOUS ZONE STAGE II DENSITY AND COMPACTION TEST SUMMARIES</p>	
Reviewed by	AS SHOWN	DESIGNED BY
Checked by		DESIGNED BY
Approved by		DESIGNED BY
		RBL-3-1475

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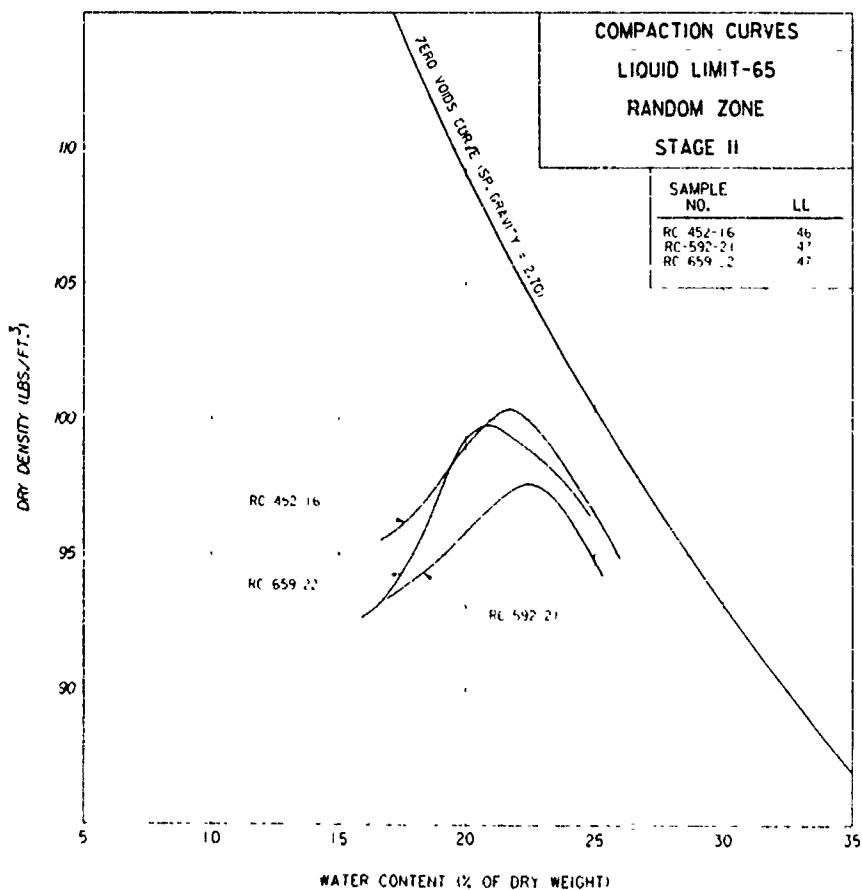
VALUE

## LIQUID LIMIT-65 RANDOM ZONE, STAGE II

RECORD CONTROL SAMPLE				SOIL CLASSIFICATION DATA						STANDARD COMPACTION TEST					
				FIELD			M.R.D. LABORATORY			FIELD			M.R.D. LAB		
NUMBER	STATION	RANGE	ELEV.	LL	PI	CLASS	SHEAR TEST SPECIMEN			COMPACTION TEST SPECIMEN			MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DR. DEN. (P.C.F.)
							LL	PI	CLASS	LL	PI	CLASS			
RC-452-16	91+20	0+60d/s	822.0	53	29	CH	54	38	CH	46	31		100.4	24.1	91.6
RC-592-21	95+10	1+10d/s	825.5	54	33	CH	62	46	CH	47	33		101.0	22.8	97.5
RC-659-22	95+70	2+60d/s	849.0	50	32	CH	57	41	CH	47	31	CL	99.5	22.0	99.7

\* UNDISTURBED DATA OBTAINED BY AVERAGING MOISTURE AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS.

NOTE: ALL DRY DENSITIES AND WATER CONTENTS ARE COMPARED TO M.R.D. DATA TO OBTAIN PERCENT COMPACTION AND WATER CONTENT PERCENT + OPTIMUM.



THE COMPACTION CURVES ARE BASED ON MISSOURI HIGHWAY DIVISION LABORATORY STANDARD COMPACTION TESTS

LIQUID LIMIT-65 RANDOM ZONE STAGE II				DENSITY	
				K.C.D. LAB	
SAMPLE	DEPTH	W (%)	DRY DENSITY (LBS./FT. <sup>3</sup> )		
RC 452-16		23.6	97.6		
RC 592-21		22.4	97.7		
RC 659-22		19.2	96.3		
PVE 13-12	5.4 5.9				
PVE 35-1	18.8 19.3				

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ALUE

## VALUE ENGINEERING PAYS

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## STANDARD COMPACTION TEST

## DENSITY AND MOISTURE CONTENT

FIELD

M.R.D. LAB

SAND CONE DATA

\* UNDISTURBED DATA

DR DEN. (P.C.F.)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	COM- PACTION (%)	WATER CONTENT (%)	WATER CONTENT (+/-20PT)	DRY DENSITY (P.C.F.)	COM- PACTION (%)	WATER CONTENT (%)	WATER CONTENT (+/-20PT)
9	92.4	24.1	100.2	20.6	97.6	97.4	23.6	+2.0	104.9	94.7	2.4	
9	92.0	22.8	97.5	22.6	97.3	99.3	22.4	0.2	97.8	99.7	2.4	
9	90.5	22.0	99.7	20.8	99.7	99.0	20.3	-0.5	99.9	99.0	2.4	

ENSIT

## DENSITY TEST SUMMARY

LIQUID LIMIT-65 RANDOM ZONE STAGE II		K.C.D. LAB					M.R.D. LAB				
AMPLE	DEPTH	W	DRY DENSITY (LBS./FT. 3)	L.L.	P.L.	P.I.	W	DRY DENSITY (LBS./FT. 3)	L.L.	P.L.	P.I.
		11.6	97.6	53	24	79	21.4	11.4	54	24	79
		2.4	97				24.7	37.2	62	24	79
		1	10.3	52	20	80	20.5	31.4	5	24	79
P.L. 11	5.4 ft.						25.1	47.4			
P.L. 15	9.4 ft.						10.9	44.9			

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

LITTLE BLUE RIVER MISSOURI  
LONGVIEW LAKE

EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
LIQUID LIMIT-65 RANDOM ZONE  
STAGE II DENSITY AND  
COMPACTION TEST SUMMARIES

R.C.D.

R.A.C.

A.L.P.

R.C.D.

US Army Corps  
of Engineers

AS SHOWN

REQ

NO.

Sheet

Number

Plot

Sheet

Design

Title

File

No.

RBL-3-1476

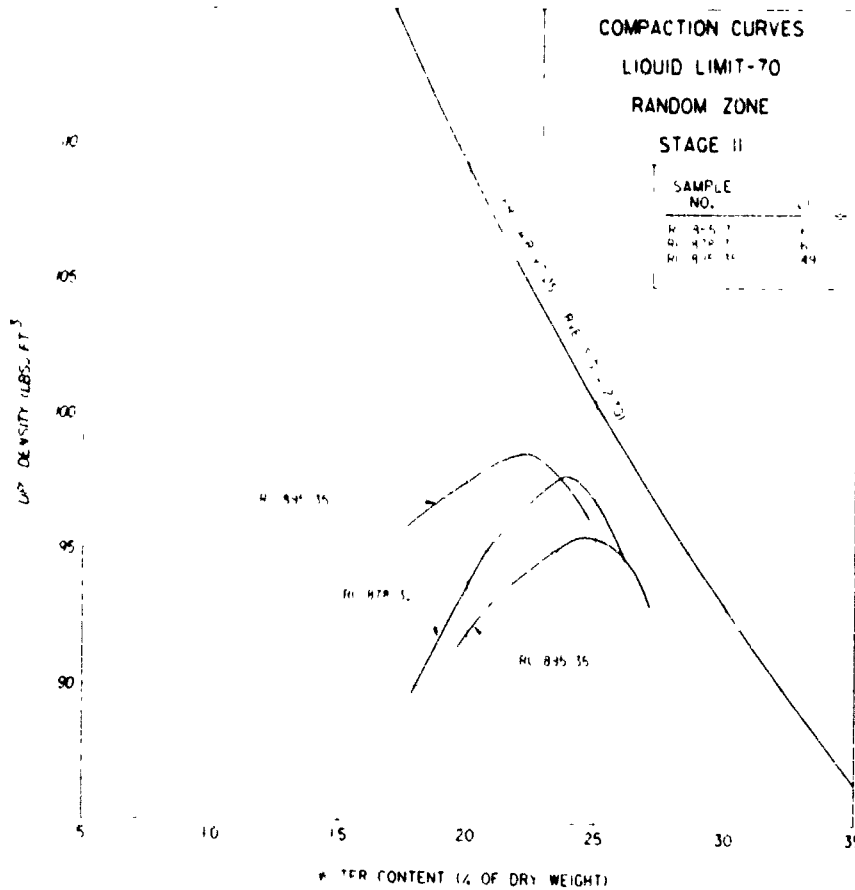
PLATE NO. 76

## LIQUID LIMIT-70 RANDOM ZONE, STAGE II

RECORD CONTROL SAMPLE				SOIL CLASSIFICATION DATA									STANDARD COMPACTION TEST				
				FIELD			M.R.D. LABORATORY						FIELD		M.R.D. LAB		
NUMBER	STATION	RANGE	ELEV.	LL	PI	CLASS	SHEAR TEST SPECIMEN			COMPACTION TEST SPECIMEN			MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)
							LL	PI	CLASS	LL	PI	CLASS					
RC 855 34	94+00	0+300 8	909	65	41	CH	62	43	CH	62	46	CH	95.5	26.0	95.6	24.8	97.5
RC 878 32	86+51	0+220 5	909	61	40	CH	59	40	CH	60	44	CH	95.5	25.5	97.7	23.7	100.6
RC 895 35	102+90	0+900 5	114	51	35	CH				49	46	CL	102.6	20.6	98.7	22.5	109.0

\* UNDISTURBED DATA OBTAINED BY AVERAGING MOISTURE AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS

NOTE: ALL DRY DENSITIES AND WATER CONTENTS ARE COMPARED TO M.R.D. DATA TO OBTAIN PERCENT COMPACTION AND WATER CONTENT PERCENT - OPTIMUM



LIQUID LIMIT-70 RANDOM ZONE STAGE II		DENSITY	
SAMPLE	DEPTH	W (%)	DRY DENSITY (LBS./FT.³)
RC 855 34		24.3	97.5
RC 878 32		24.0	100.6
RC 895 35	13.8-14.3		
RC 895 35	0.0-0.5		

THE COMPACTION CURVES ARE BASED ON M. R. D. DIVISION LABORATORY STANDARD COMPACTION TEST

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STANDARD COMPACTION TEST					DENSITY AND MOISTURE CONTENT							
FIELD		M.R.D. LAB			SAND CONE DATA				* UNDISTURBED DATA			
NO.	DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-%OPT)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-%OPT)
15	95.5	26.0	95.6	24.8	97.5	102.0	24.9	+0.1	104.0	108.8	22.5	-2.3
16	95.5	25.5	97.7	23.7	100.6	103.0	24.0	+0.3	100.7	103.1	24.4	+0.7
17	92.6	25.6	98.7	22.5	109.0	101.4	26.2	-2.3				

DENSITY TEST SUMMARY											
LIQUID LIMIT-70 RANDOM ZONE STAGE II		M.R.D. LAB					M.P.D. LAB				
SAMPLE	DEPTH	W	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.	W	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.I.
15		24.9	97.5	65	24		22.5	104.0	61	24	
16		24.0	100.6	61			24.4	100.7	59		
17	3.9-4.3						24.4	98.0			
18	4.0-5.5						25.3	97.2			

Symbol	Revision	Date	App. Cond.
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Prepared by	R.G.D.	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT LIQUID LIMIT-70 RANDOM ZONE STAGE II DENSITY AND COMPACTION TEST SUMMARIES	
Checked by	R.A.C.		
Approved by	A.L.P.	AS SHOWN	
Reviewed by	R.G.D.		
		RBL-3-1477	

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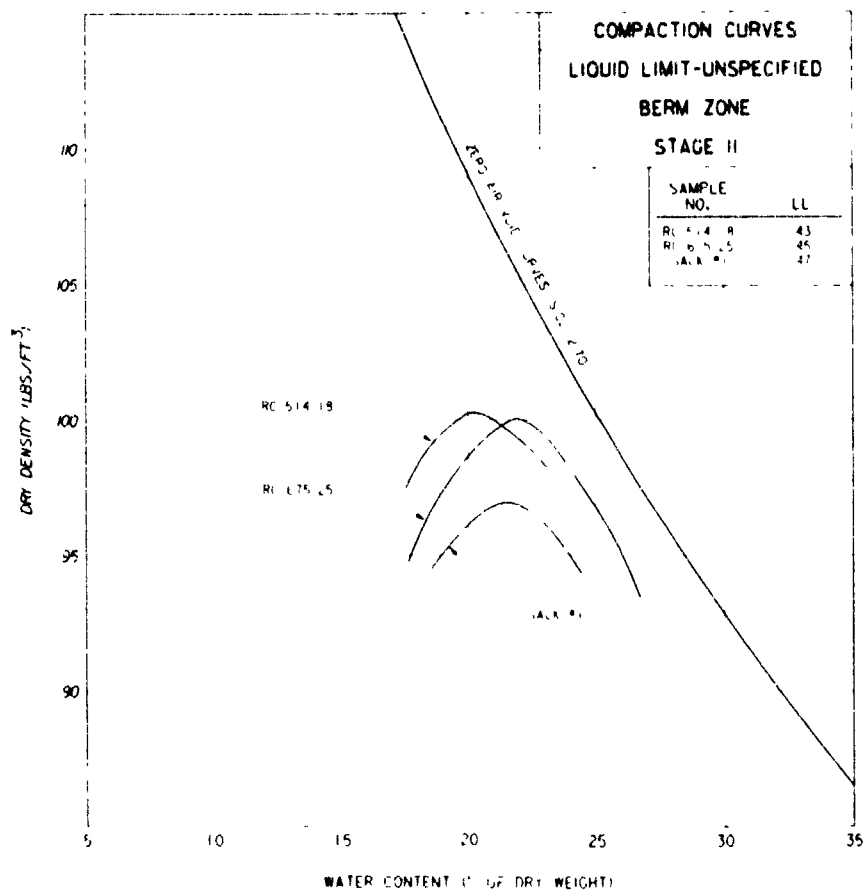
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LIQUID LIMIT-UNSPECIFIED BERM ZONE, STAGE II

RECORD CONTROL SAMPLE										SOIL CLASSIFICATION DATA			STANDARD COMPACTION TEST				
										FIELD			M.J.D. LAB		SAND		
NUMBER	STATION	RANGE	ELEV.	LL	PI	CLASS	SHEAR TEST SPECIMEN			COMPACTION TEST SPECIMEN			MAX. DRY DENSITY	OPT. MOIST. CONT.	MAX. DRY DENSITY	OPT. MOIST. CONT.	DRY DENSITY
							NO.	PI	CLASS	LL	PI	CLASS	(P.C.F.)	(%)	(P.C.F.)	(%)	(P.C.F.)
RC 54 18	96+56	2+350.8	853.5	47	11	CL	49	33	CL	43	16	CL	119	14	100.3	17	119
RC 675 25	93+30	2+200.8	864.0	55	33	CH	58	43	CH	45	17	CH	114	18	99.9	20.8	114
SACK #1	RIGHT	ABUT.	SEEP				47	29	LL	47	29	LL			96.8	21.7	

\* UNDISTURBED DATA OBTAINED BY AVERAGING MOISTURE AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS.  
NOTE: ALL DRY DENSITIES AND WATER CONTENTS ARE COMPARED TO M.J.D. DATA TO OBTAIN PERCENT COMPACTION AND WATER CONTENT PERCENT % OF MAX.



THE COMPACTION CURVES ARE BASED ON MOISTURE AND DENSITY VALUES OBTAINED BY STANDARD COMPACTION TESTS.

LIQUID LIMIT-UNSPECIFIED BERM ZONE		FIELD LAB	
STATION	DEPTH	W	DRY DENSITY
96+56	1.5	14	119
93+30	1.5	18	114
96+56	1.5	14	119
93+30	1.5	18	114



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## STANDARD COMPACTION TEST

## DENSITY AND MOISTURE CONTENT

SAND	FIELD	M.R.D. LAB				SAND CONE DATA				*UNDISTURBED DATA			
		DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	MAX. DRY DENSITY (P.C.F.)	OPT. MOIST. CONT. (%)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-%OPT)	DRY DENSITY (P.C.F.)	COMPACTION (%)	WATER CONTENT (%)	WATER CONTENT (+-%OPT)
A	2.5	104.7	19.3	105.0	19.3	104.7	98.0	22.7	+0.8	104.7	99.4	19.3	+0.6
4	2.8	99.9	21.8	98.0	21.8	98.0	98.0	22.7	+0.9	99.4	99.5	21.2	-0.6
		96.8	21.7										

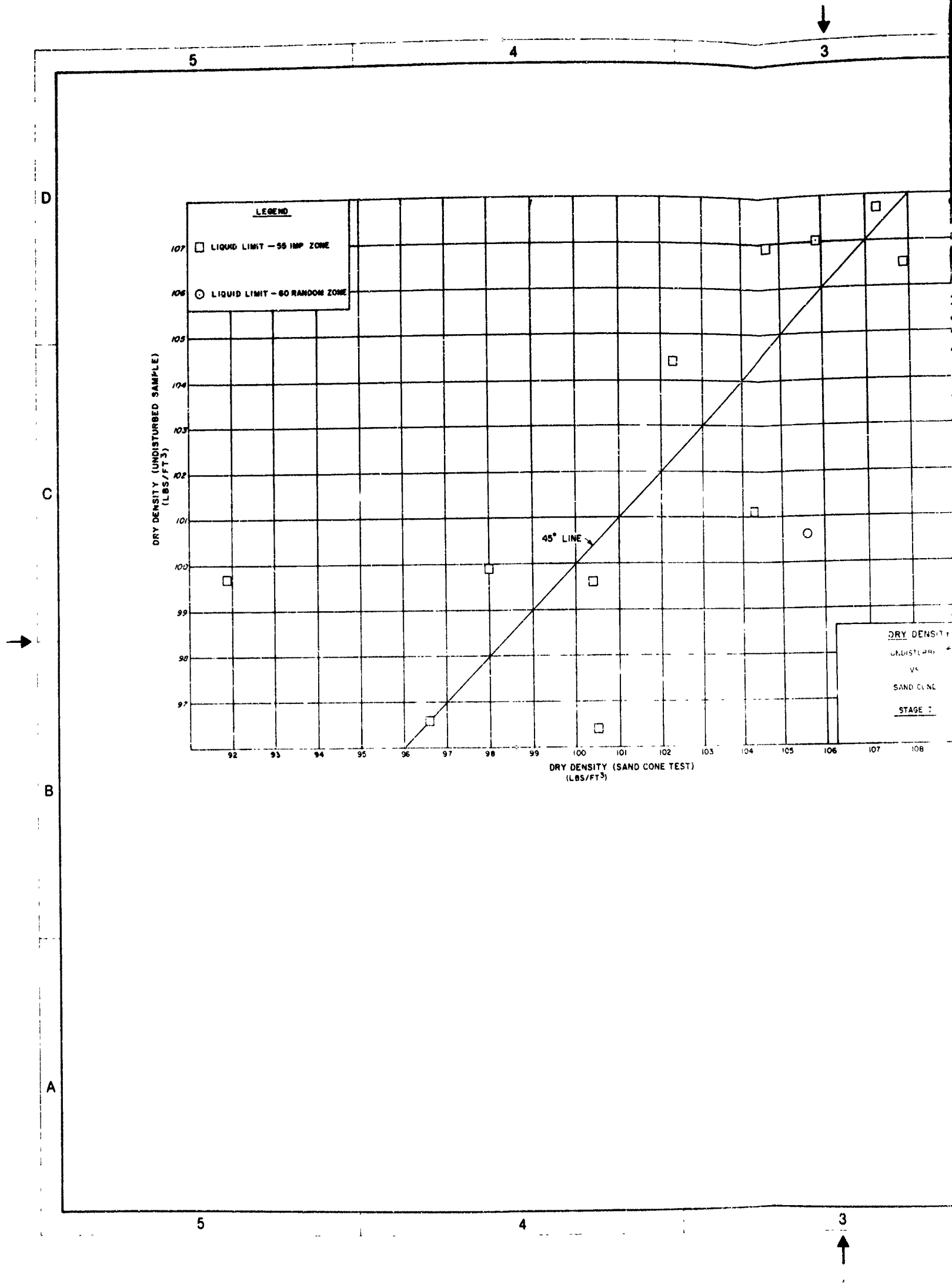
## DENSITY TEST SUMMARY

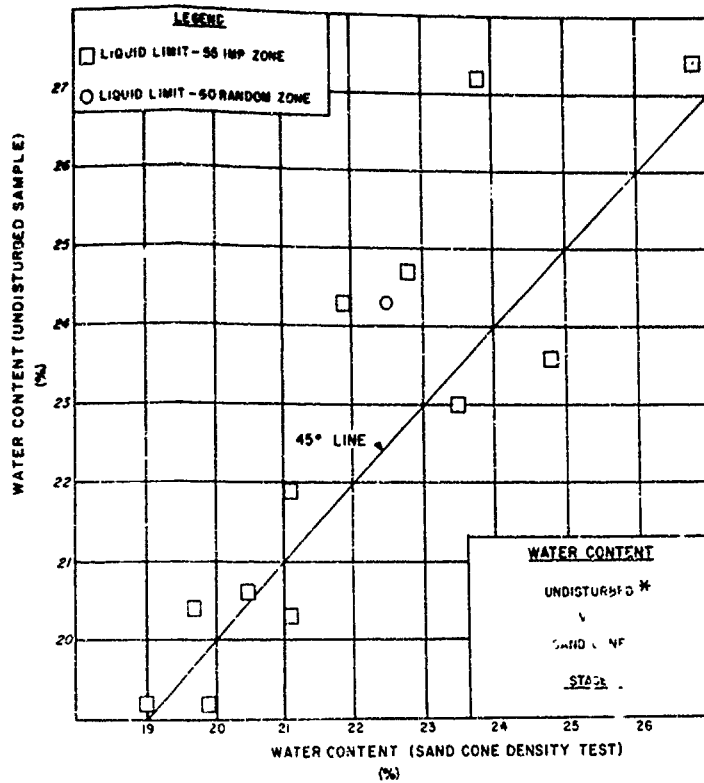
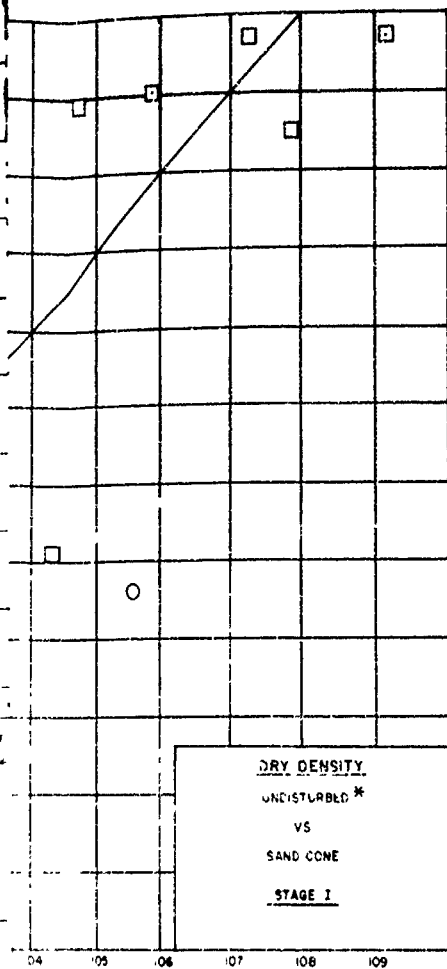
M.T. UNSPEC. ZONE		P.C.D. LAB			M.R.D. LAB					
					HEAVY COMPACTION METHOD					
DEPTH		DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.L.	W (%)	DRY DENSITY (LBS/FT <sup>3</sup> )	L.L.	P.L.	P.L.
5		51	22	30	19.3	104.7	49	16	13	
4		54	20	34	2	99.4	58	15	43	

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURILITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
BERM ZONE (NO SPECIFIED LIQUID LIMIT)  
STAGE II DENSITY AND  
COMPACTION TEST SUMMARIES

RBL-3-1478

PLATE NO. 78





\*NOTE  
THE UNDISTURBED DATA WAS OBTAINED BY AVERAGING  
MOISTURE AND DENSITY VALUES OF ALL AVAILABLE  
SHEAR TEST SPECIMENS FOR EACH RECORD CONTROL  
SAMPLE

Revisions		Date	Approved
Symbol	Descriptions		

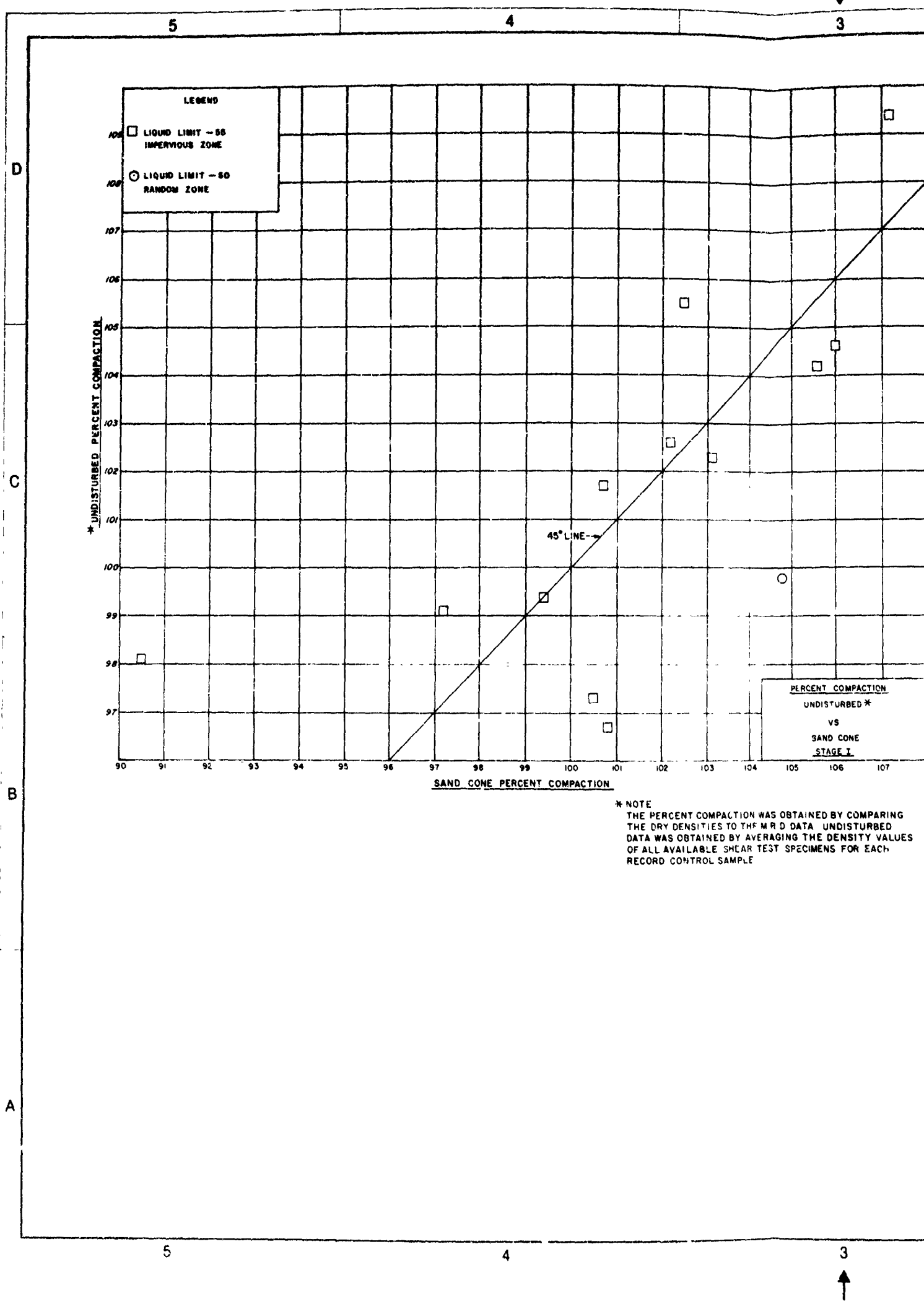
U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: W.B.E.  
Drawn by: R.A.C.  
Checked by: W.B.E./R.G.D.  
Submitted by: F.C.W.

**LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT**

**COMPACTION TEST SUMMARIES  
STAGE I**

Scale: \_\_\_\_\_  
Date: \_\_\_\_\_  
Sheet Number: \_\_\_\_\_  
File No: RBL-3-1479



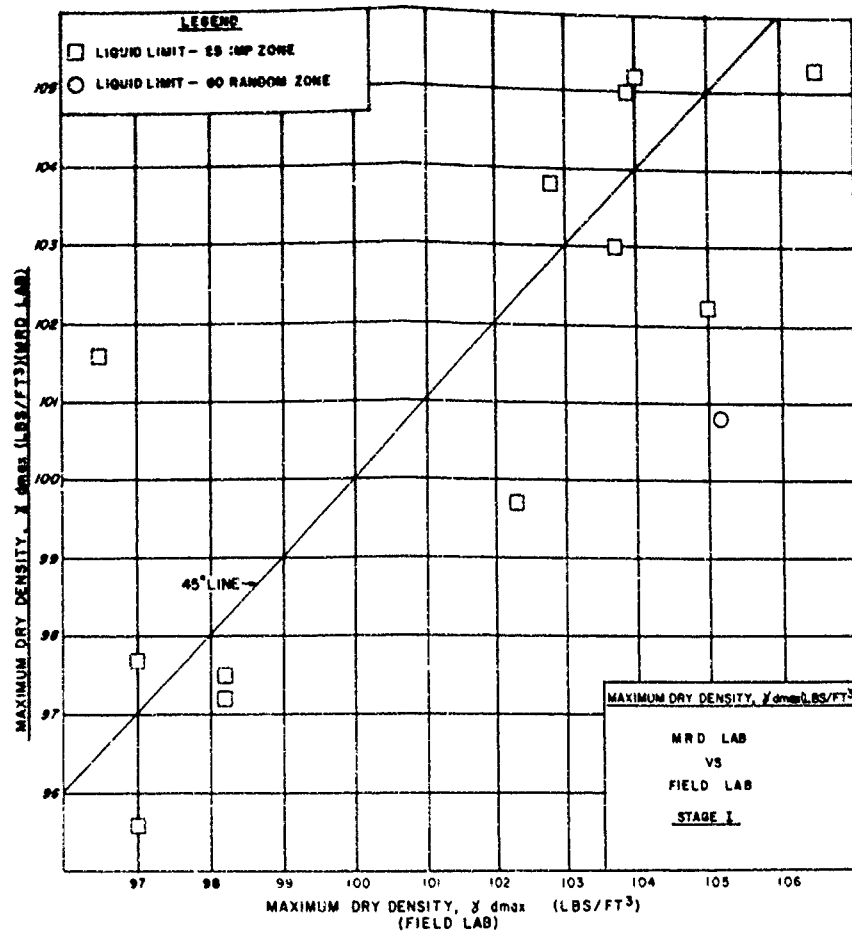
OPTIMUM WATER CONTENT, % (M R D LAB)

24  
23  
22  
21  
20  
19  
18

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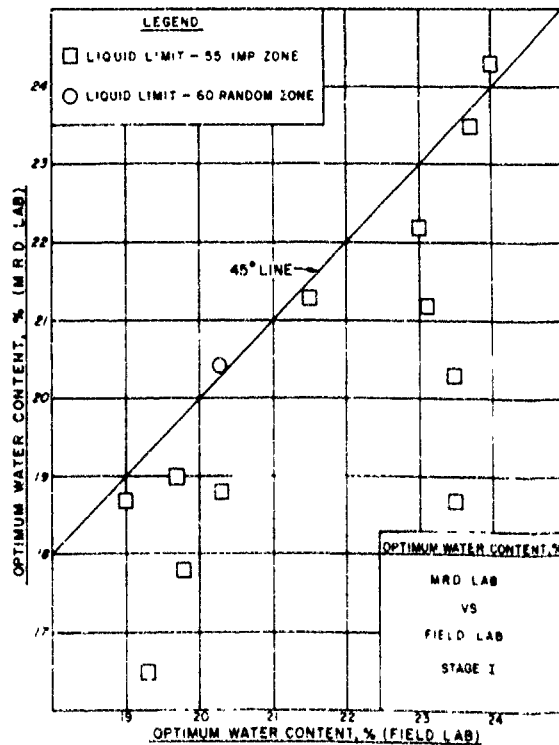


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Symbol	Revisions	Descriptions	Date	Approved

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

COMPACTION TEST SUMMARIES  
STAGE I

WBE  
JGS  
WBE/RGD  
FCW

AS SHOWN

RBL-3-1480

PLATE NO 80

3

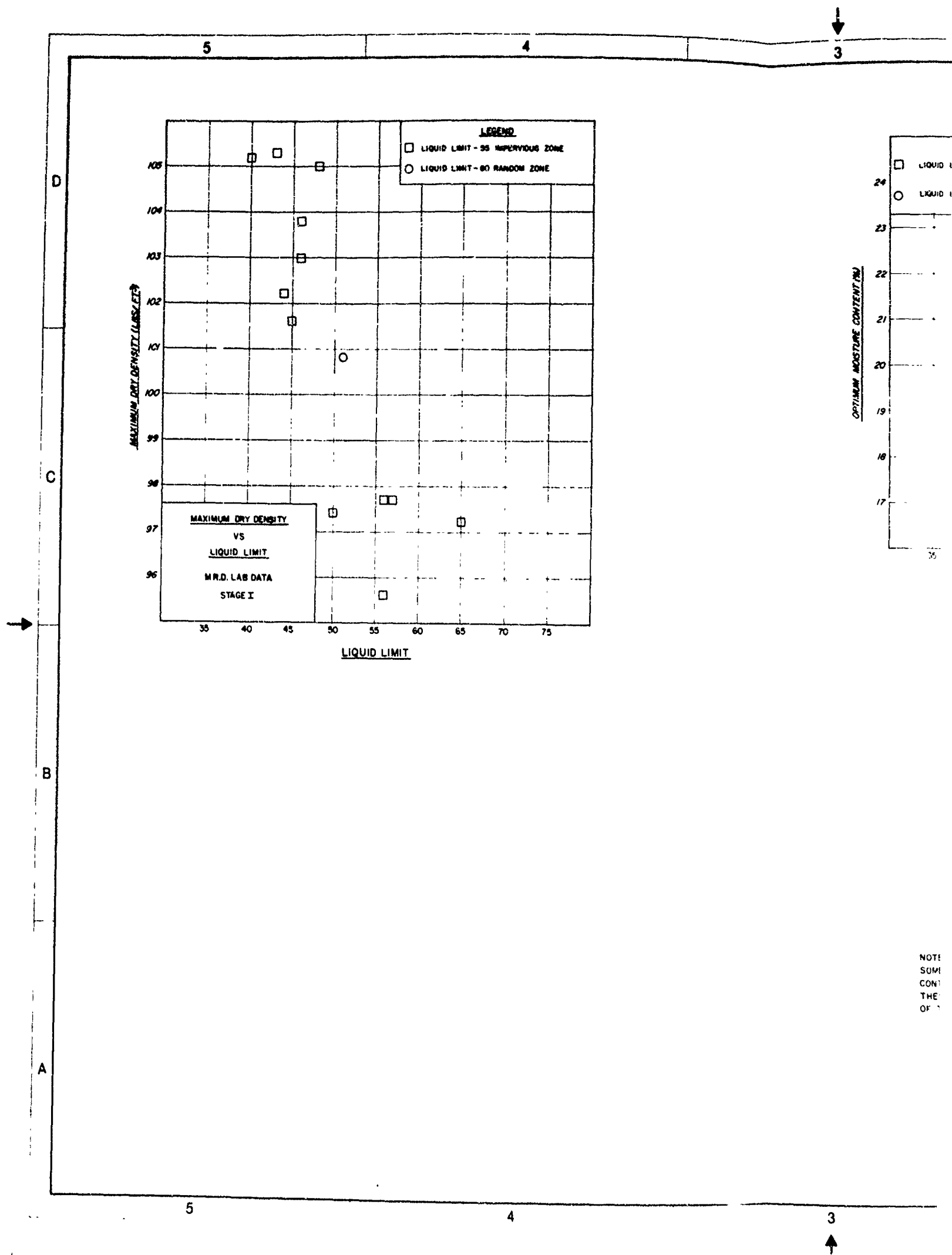
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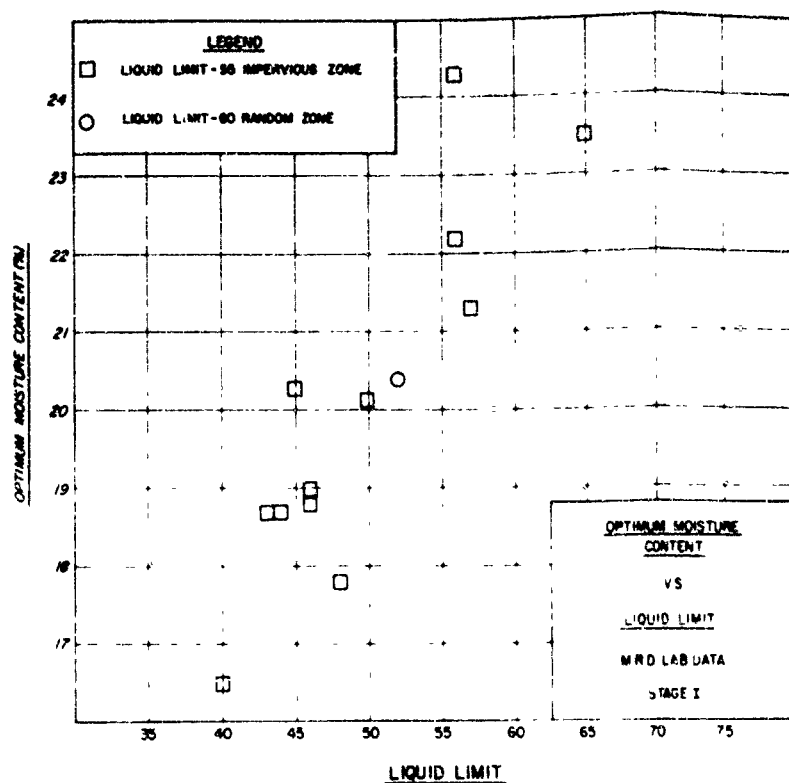
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PERCENT COMPACTION  
UNDISTURBED \*  
VS  
SAND CONE  
STAGE I

CTION WAS OBTAINED BY COMPARING  
TO THE MRD DATA UNDISTURBED  
BY AVERAGING THE DENSITY VALUES  
NEAR TEST SPECIMENS FOR EACH  
MPLE

OPTIMUM WATER CONTENT, % (MRD LAB)





NOTE:  
SOME OF THE IMPERVIOUS ZONE LIQUID LIMIT 55 RECORD  
CONTROL SAMPLES HAVE A LIQUID LIMIT GREATER THAN 55  
THESE SAMPLES WERE LOCATED IN THE LOWER PORTION  
OF THE CUTOFF TRENCH

Revisions		Date	Approved
Symbol	Descriptions		

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Described by: WBE

Drawn by: SEV

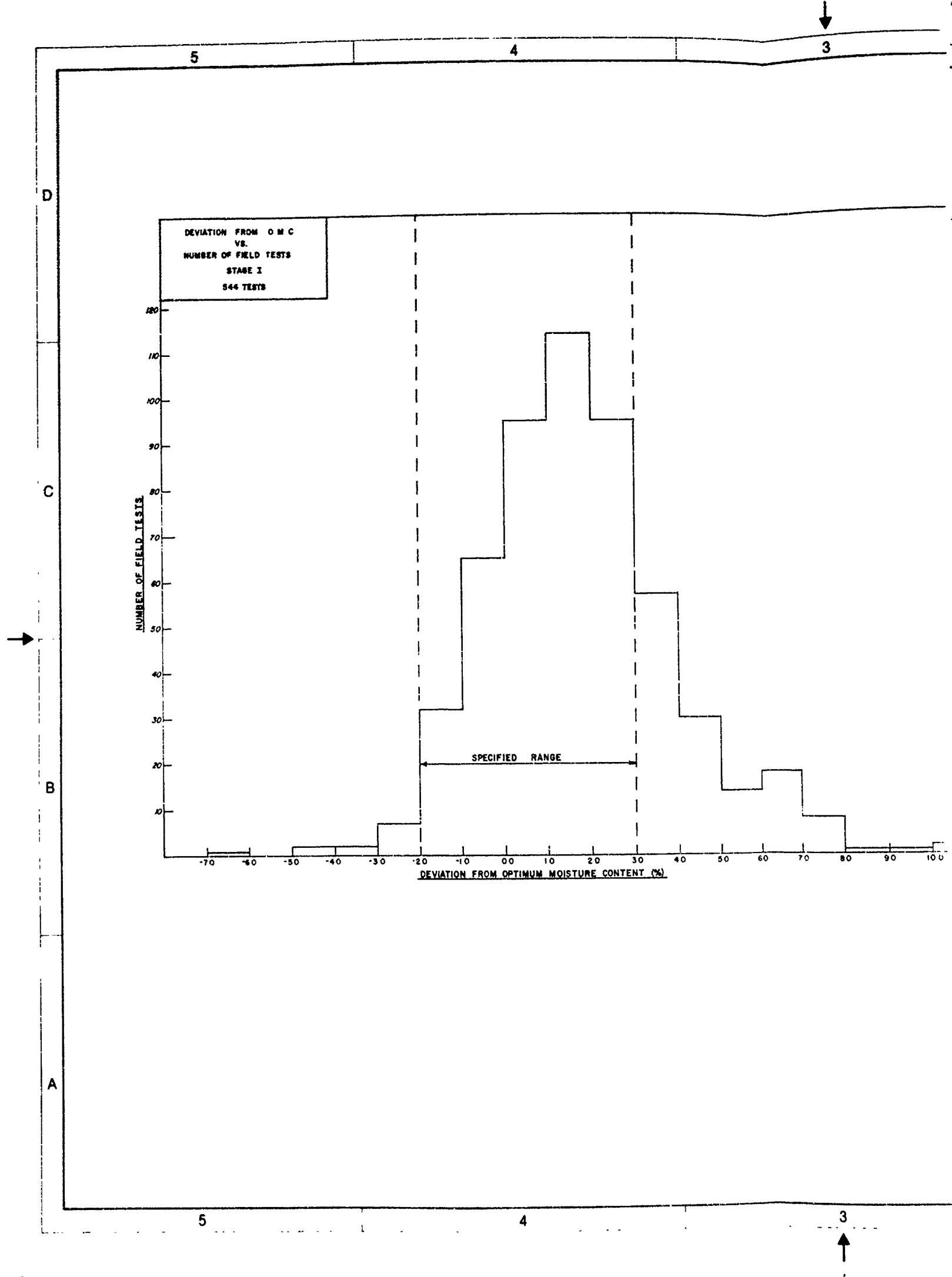
Revised by: RGD/WBE

Submitted by: FCW

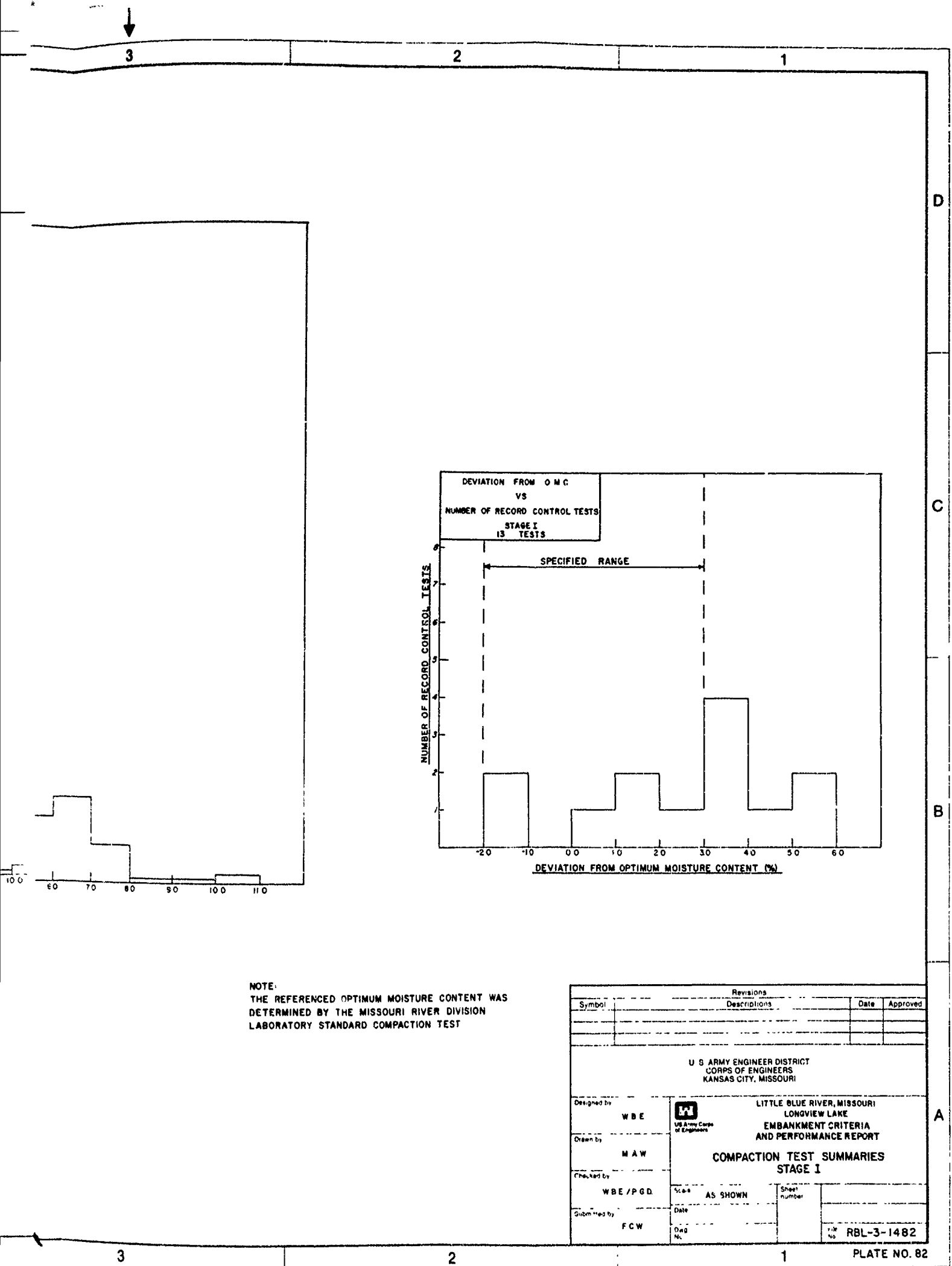
AS SHOWN

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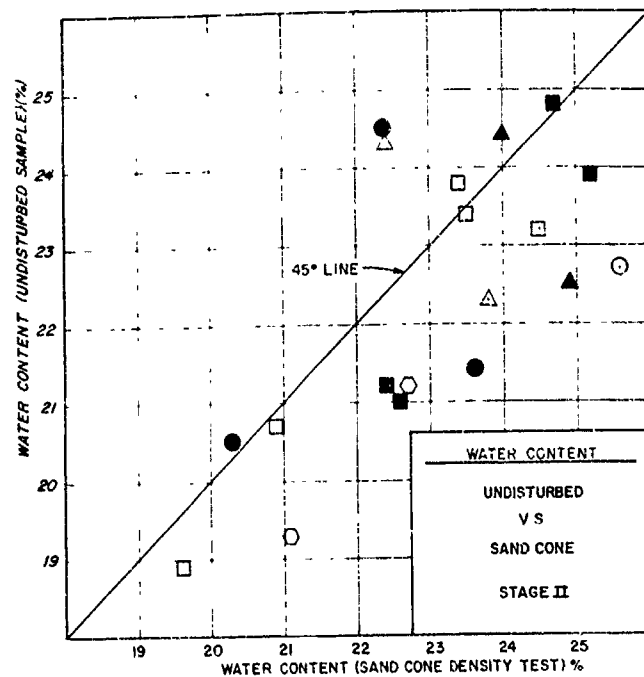
PLATE NO. 81



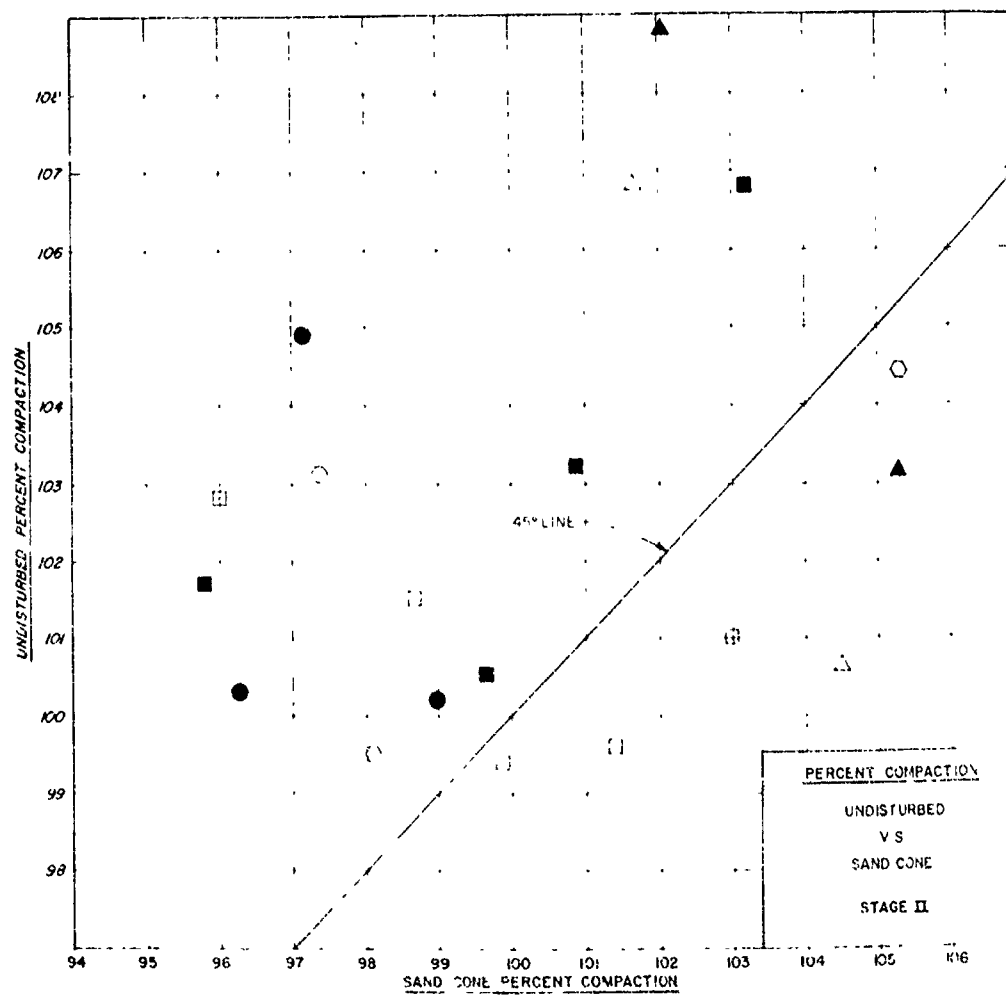




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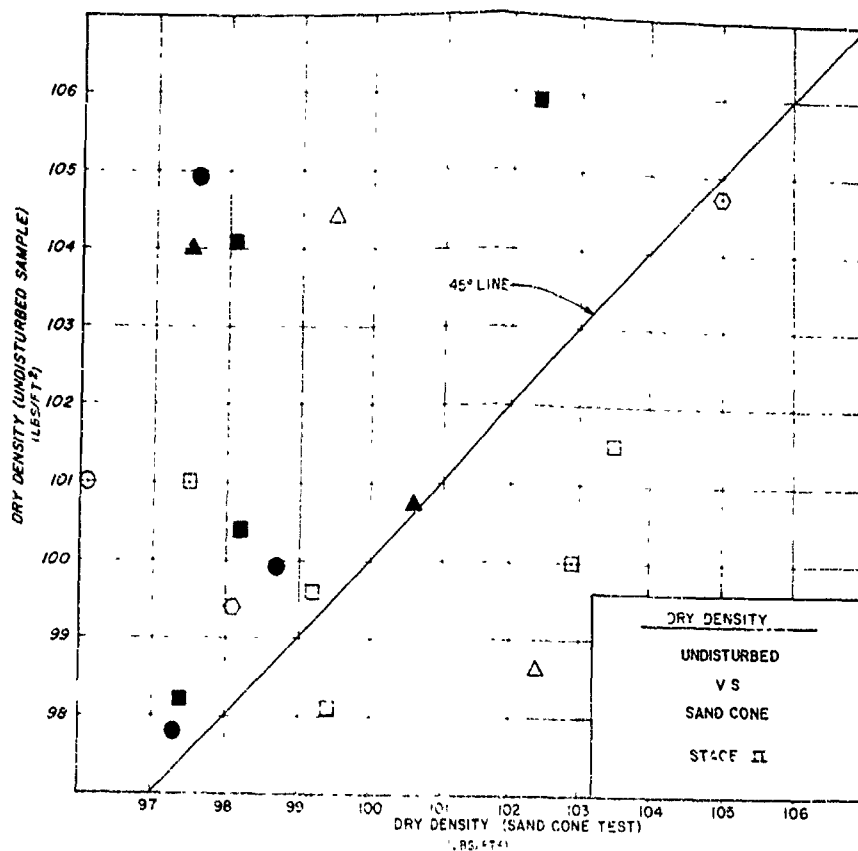
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WATER CONTENT (UNDISTURBED SAMPLE)

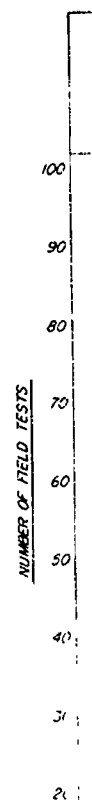
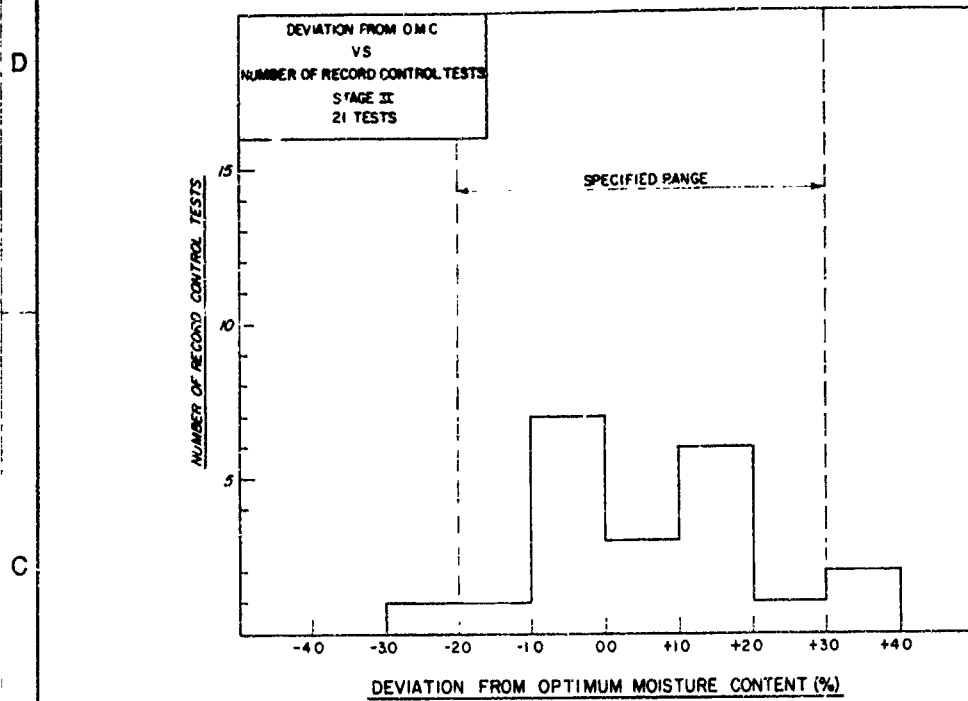
7-27 F.V. (UNDISTURBED SAMPLE)



LEGEND - STAGE II		
SYMBOL	SAMPLE	LOCATION
□	LIQUID LIMIT	55 IMP ZONE
○	LIQUID LIMIT	60 IMP ZONE
△	LIQUID LIMIT	60 RDM ZONE
■	LIQUID LIMIT	65 IMP ZONE
●	LIQUID LIMIT	65 RDM ZONE
▲	LIQUID LIMIT	70 RDM ZONE
◇	BERM ZONE	UNSPEC. LIQUID LIMIT

NOTE  
THE UNDISTURBED DATA WAS OBTAINED BY AVERAGING MOISTURE  
AND DENSITY VALUES OF ALL AVAILABLE SHEAR TEST SPECIMENS  
FOR EACH RECORD CONTROL SAMPLE

Symbol	Revisions	Date	Approved
+	Description		
+			
U.S. ARMY ENGINEER DISTRICT CHIEFS OF ENGINEERS KANSAS CITY, MISSOURI			
LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT COMPACTION TEST DATA SUMMARY STAGE II			
WBE	Scale	As Shown	Sheet
EGM	Date		
RGD-WBE	Tag		
FCW	No.		
			File No. RBL-3-1483



NOTE  
THE REFERENCE  
BY MISSOURI RIV

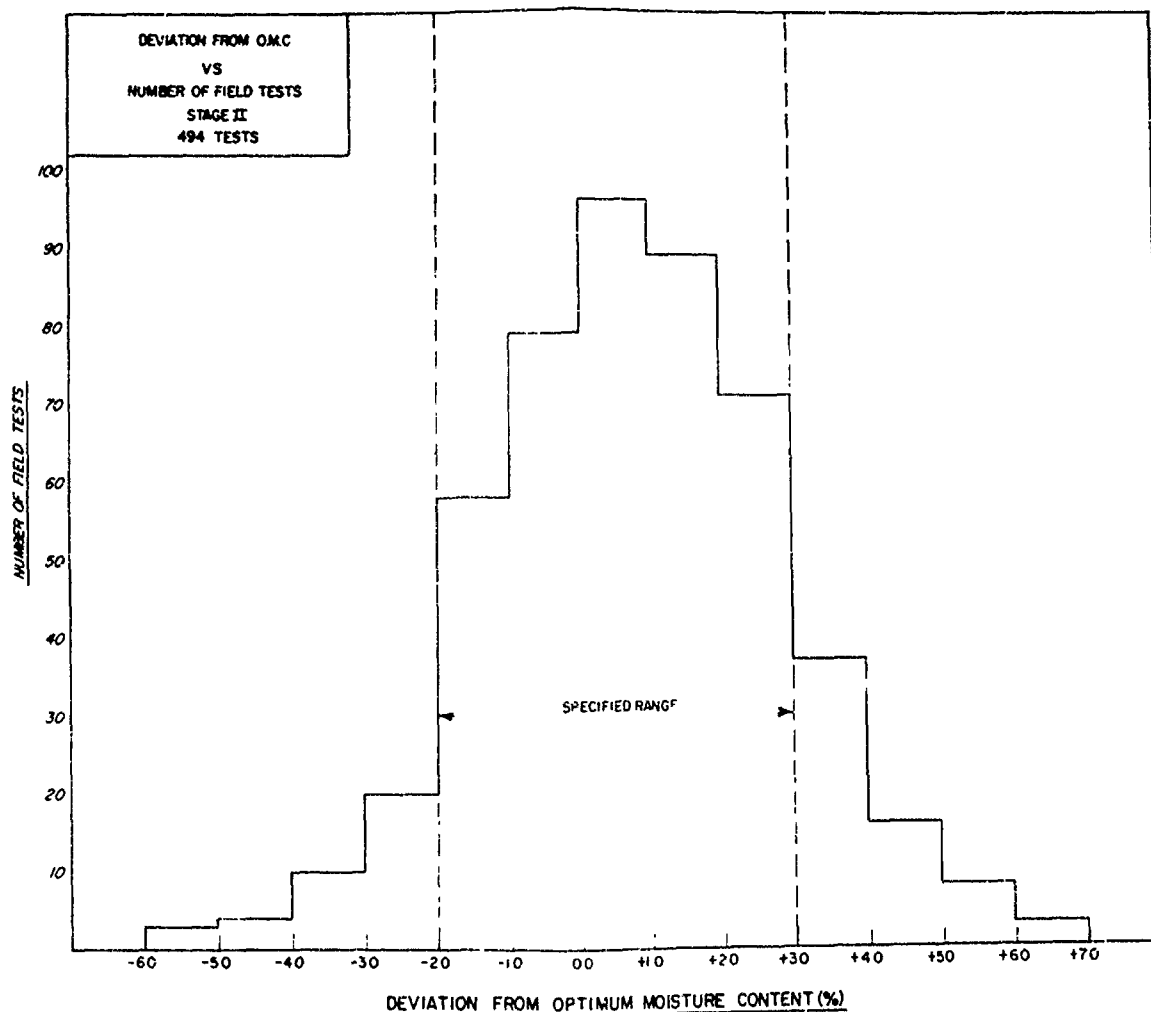
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NOTE  
THE REFERENCED OPTIMUM MOISTURE CONTENT WAS DETERMINED  
BY MISSOURI RIVER DIVISION LABORATORY STANDARD COMPACTION TEST

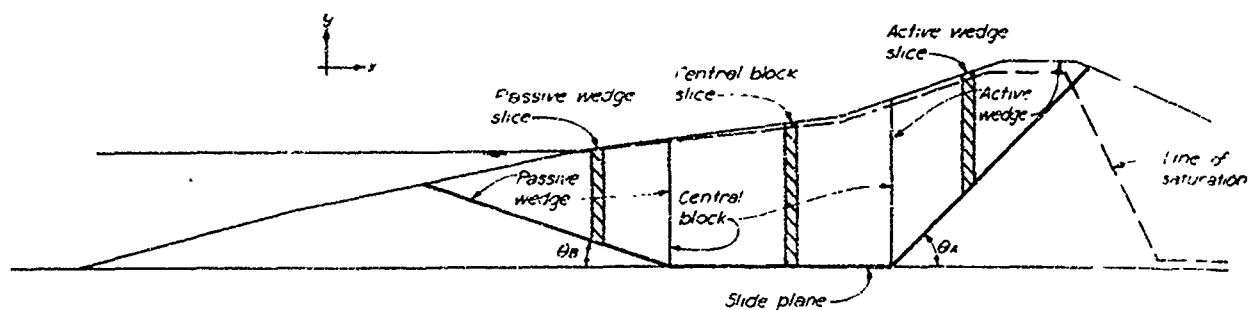
Revisions		Date	Approved
Symbol	Descriptions		

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: WBE  
DRAWN BY: SEY  
CHECKED BY: WBE/RGD  
SUBMITTED BY: FCW

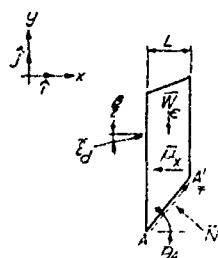
LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT  
COMPACTION TEST SUMMARIES  
STAGE II

Scale: AS SHOWN  
Date:   
Sheet number: 3 of 3  
File No: RBL-3-1485



GENERALIZED CROSS-SECTION SHOWING A TYPICAL SLIDE PLANE

### ACTIVE WEDGE ANALYSIS



$$\begin{aligned} W &= W_0 \quad (1a) \\ W_x &= W_0 \quad (1b) \\ N &= W \sin \theta_B \quad (1c) \\ T &= W \cos \theta_B \quad (1d) \\ T &= L \sin \theta_B \quad (1e) \end{aligned}$$

For the above freebody to be in equilibrium the following condition must be satisfied:

$$\sum F = 0 \quad (2)$$

But  $N$  can be represented as,

$$N = A \sin \theta_B$$

where,

$A$  = area of slice  $\times$  unit width  $\times$  unit height  
 $= L \cos \theta_B$   
 $\theta_B$  = average bearing angle of slice

Therefore,

$$N = A \sin \theta_B \quad (3)$$

Similarly,  $T$  can be represented as

$$T = A \cos \theta_B$$

where  $A$  is defined as above.

$$T = A \cos \theta_B$$

with

$F.S.$  = Factor of safety against sliding  
 $T$  = Average shear stress acting on slice

From Coulomb's Law

$$T = c + \sigma \tan \phi$$

where  $c$  and  $\tan \phi$  are shear strength parameters.  
 Therefore,

$$\frac{c}{F.S.} + \frac{\sigma \tan \phi}{F.S. \cos \theta_B} = T \quad (4)$$

Substituting Equations (1a), (1b), (1c), (1d), and (1e) into Equation (2), and expressing in matrix form the condition that the algebraic summation of the coefficients of  $T$  and  $N$  equals zero yields

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} \begin{Bmatrix} A_1 \\ A_2 \end{Bmatrix} \quad (5)$$

where

$$\begin{Bmatrix} A_1 \\ A_2 \end{Bmatrix} = \begin{Bmatrix} \cos \theta_B \\ \sin \theta_B \end{Bmatrix} \quad (6)$$

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} = \begin{Bmatrix} 1 \\ -F.S. \tan \theta_B \end{Bmatrix}$$

The solution to Equation (5) is

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} \begin{Bmatrix} A_1 \\ A_2 \end{Bmatrix} \quad (7)$$

where

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} = \begin{Bmatrix} 1 \\ -F.S. \tan \theta_B \end{Bmatrix} \quad (8)$$

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} = \begin{Bmatrix} 1 \\ -F.S. \tan \theta_B \end{Bmatrix} \quad (9)$$

Using the solution for  $T$  given by Equation (4) and substituting it into Equation (3), the final condition of the design with respect to equilibrium for the slice is

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} \begin{Bmatrix} A_1 \\ A_2 \end{Bmatrix} \quad (10)$$

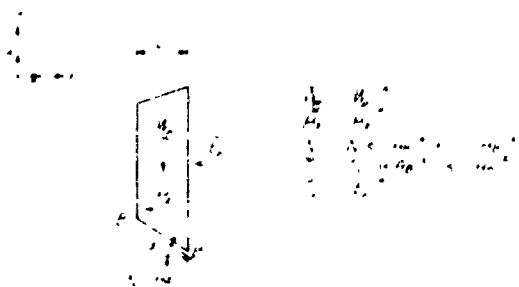
where

$$\begin{Bmatrix} F_1 \\ F_2 \end{Bmatrix} = \begin{Bmatrix} 1 \\ -F.S. \tan \theta_B \end{Bmatrix} \quad (11)$$

# DEFINITION OF SYMBOLS USED IN DERIVATION OF STABILITY EQUATIONS

- $\hat{i}$  unit vector in x direction
- $\hat{j}$  unit vector in y direction
- $\vec{W}$  Effective weight vector
- $\vec{H}$  Net horizontal seepage force vector (equal to the vector summation of the horizontal components of all water forces acting on the slice)
- $N$  Normal force vector acting perpendicular to slide surface
- $T$  Tangential force vector acting tangentially to slide surface
- $\vec{D}$  Driving earth force vector contributed by active wedge
- $\vec{R}$  Resisting earth force vector contributed by passive wedge
- $\vec{C}$  Central block force
- $\vec{M}$  Moment vector
- $\theta$  Slope of driving earth force vector

## PASSIVE WEDGE ANALYSIS



The passive wedge analysis is a method for determining the stability of a soil mass. It involves analyzing the forces acting on a vertical slice of soil, including weight, seepage force, normal force, tangential force, driving force, and resisting force. The slice is shown in a state of equilibrium.

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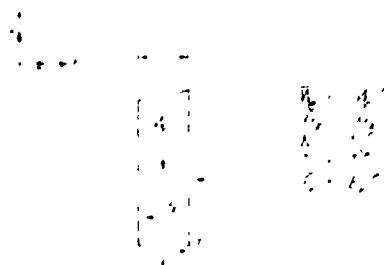
The passive wedge analysis is a method for determining the stability of a soil mass. It involves analyzing the forces acting on a vertical slice of soil, including weight, seepage force, normal force, tangential force, driving force, and resisting force. The slice is shown in a state of equilibrium.

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## CENTRAL BLOCK ANALYSIS



The central block analysis is a method for determining the stability of a soil mass. It involves analyzing the forces acting on a central block of soil, including weight, seepage force, normal force, tangential force, driving force, and resisting force. The block is shown in a state of equilibrium.

The central block analysis is a method for determining the stability of a soil mass. It involves analyzing the forces acting on a central block of soil, including weight, seepage force, normal force, tangential force, driving force, and resisting force. The block is shown in a state of equilibrium.

SYM

DESCRIPTION  
REVISIONS

DATE APP'D

LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
EMBANKMENT CRITERIA AND PERFORMANCE REPORT  
DERIVATION OF STABILITY EQUATIONS

In 1 sheet  
CORPS OF ENGINEERS  
KANSAS CITY DISTRICT

Sheet No. 1

Scale as shown  
U. S. ARMY  
JUNE 1978

Designed by  
Checked by  
Reviewed by  
Approved by  
R. L. W.

Reviewed by  
Checked by  
Approved by  
R. L. W.

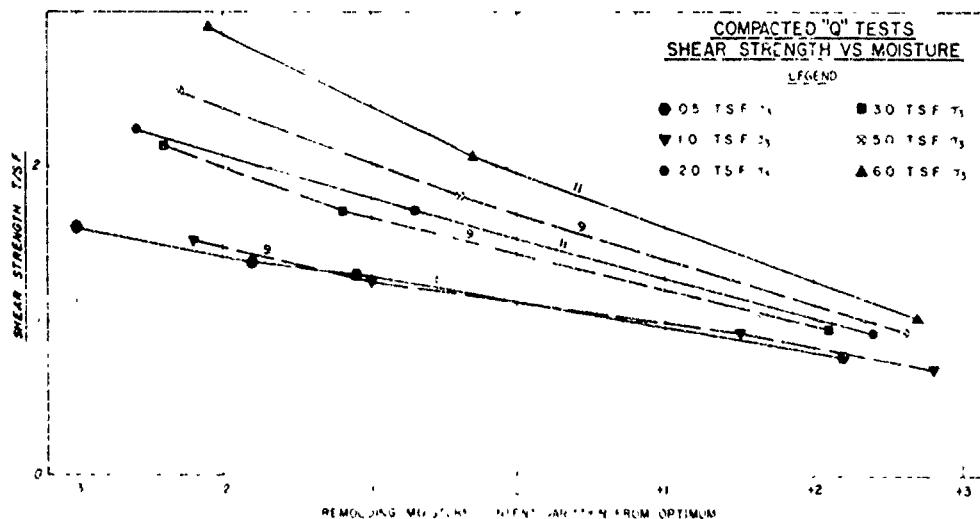
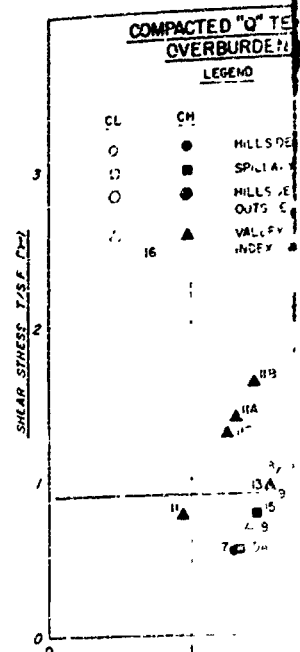
Reviewed by  
Checked by  
Approved by  
R. L. W.

RBL-3-1564

PLATE NO. 8

"Q" TEST DATA ON COMPOSITE SAMPLES													
INDEX NUMBER	COMPOSITE NUMBER	CLASSIFICATION			DRY DENSITY		WATER CONTENT		N	T	C	TAN $\phi$	LOCATION
		SYMBOL	LL	PL	PI	MAXIMUM	AT TEST	OPTIMUM					
6*	6	CL	44	17	27	95.5	90.8	18.5	21.8	1.75	1.30	100	HILLSIDE BORROW
			50		33		90.7		21.6	3.97	1.68		
							90.1		22.4	3.99	1.72		
6A*	6	CL	44	17	27	95.5	95.9	18.5	20.9	1.80	1.38	13	HILLSIDE BORROW
			50		33		95.4		21.8	3.91	1.58		
7	7	CH	50	15	40	98.5	94.4	23.5	26.1	1.32	.54	50	HILLSIDE BORROW
			52	6	36		93.6		26.3	3.32	.56		
8	8	CL	45	15	30	103.5	99.0	19.5	22.2	1.59	1.02	80	DOWNSTREAM VALLEY
							98.6		22.6	3.89	1.54		
							98.1		22.2	6.00	1.73		
9	9	CL	43	17	26	101.5	97.4	21.0	22.5	1.53	.92	60	VALLEY
							96.9		23.1	3.55	.95		
							96.7		23.6	5.54	.93		
							96.7		23.8	1.40	.69		
9A	9	CL	43	17	26	101.5	97.7	21.0	20.0	1.72	1.25	100	VALLEY
							97.3		19.8	3.99	1.71		
							96.7		20.6	6.04	1.89		
9B	9	CL	43	17	26	101.5	96.7	21.0	18.8	1.86	1.52	110	VALLEY
							96.6		18.6	4.23	2.13		
							96.9		19.7	6.46	2.48		
10	10	CL	38	18	20	105.0	99.2	17.0	19.7	1.59	1.03	85	VALLEY
							99.3		19.3	2.67	1.16		
							99.4		19.5	4.84	1.45		
11	11	CH	51	16	35	103.5	99.5	19.5	21.7	.94	.77	08	VALLEY
							98.8		21.9	2.53	.92		
							98.8		22.2	6.58	1.01		
11A	11	CH	51	6	35	103.5	100.0	10.5	17.7	1.30	1.39	13	VALLEY
							99.5		18.6	2.98	1.71		
							99.0		19.2	7.19	2.06		
11B	11	CH	51	16	35	103.5	99.3	10.5	16.5	1.43	1.61	13	VALLEY
							98.6		16.9	3.29	2.24		
							93.7		17.4	7.67	2.89		
11C	11	CH	51	16	35	103.5	99.3	19.5	18.3	1.24	1.20	14	VALLEY
13	13	CH	50	17	33	103.5	98.5	19.5	22.5	1.54	.94	92	OUTLET WORKS
							98.4		22.5	2.59	1.02		
							98.5		22.5	5.72	1.25		
15	15	CH	55	16	39	102.0	93.8	20.0	23.0	1.44	.77	75	SPILLWAY
							92.9		23.4	3.47	.81		
15A	15	CH	55	16	39	102.0	82.4	20.0	23.5	1.31	.54	75	SPILLWAY
							82.5		23.2	3.41	.71		

\* COMPOSITE NO 6 TAKEN OUTSIDE THE PROPOSED BORROW AREA





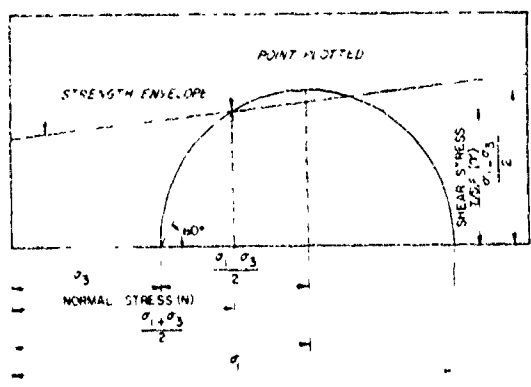
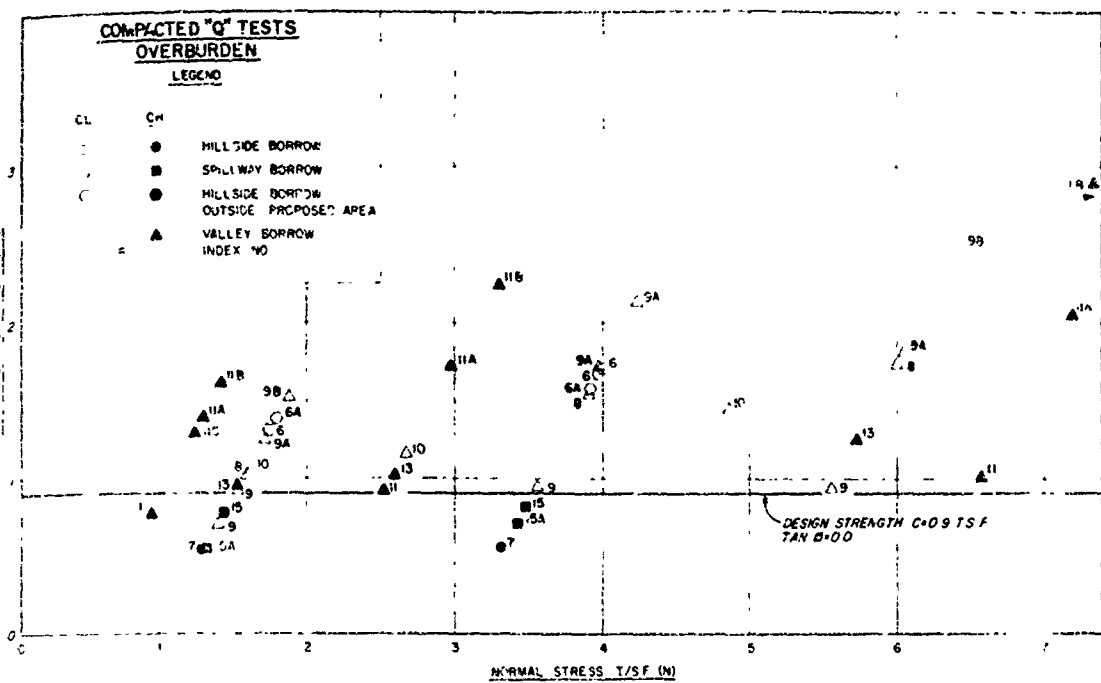
DATE  
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ULLEY  
DEF 18

**COMPACTED "Q" TESTS  
OVERBURDEN**

- LEGEND**
- CL CM
  - HILLSIDE BORROW
  - SPILLWAY BORROW
  - HILLSIDE BORROW OUTSIDE PROPOSED AREA
  - ▲ VALLEY BORROW INDEX NO

SHEAR STRESS T/SF (N)



**MOHR'S DIAGRAM**

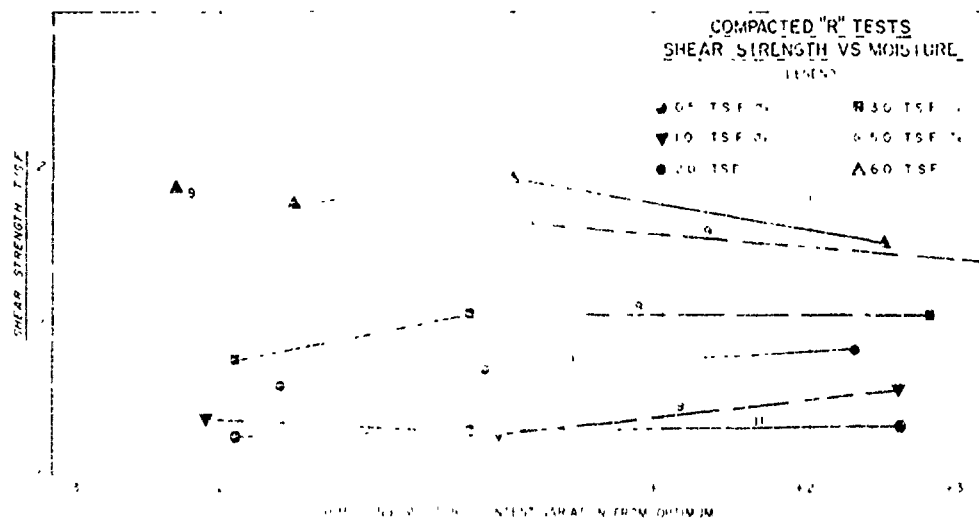
SYM DESCRIPTION REVISIONS DATE APP'D

**LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
TEST DATA SUMMARY  
COMPACTED EMBANKMENT MATERIAL  
Q-TESTS AND ATTERBERG LIMIT TESTS**

In 1 sheet Sheet No. 1 Scale as shown  
CORPS OF ENGINEERS U.S. ARMY  
KANSAS CITY DISTRICT JUNE 1978  
Submitted Recommended  
A. G. G. J. P. M. R. G. D.  
RBL-3-1686

"R" TEST DATA ON COMPOSITE SAMPLES															
INDEX	COMPOSITE	CLASSIFICATION				DRY DENSITY		WATER CONTENT		N	P	C	TAN $\phi$	LOCATION	
NUMBER	NUMBER	SYMBOL	LL	PL	PI	MAXIMUM	AT TEST	OPTIMUM	AT TEST	T S F	T S F	T S F	TAN $\phi$	LOCATION	
6*	6	CL	44	17	27	95.5	91.7 90.7	18.5	21.0 21.7	129 358	50 100	25	22	HILLSIDE BORROW	
6A*	6	CL	44	17	27	95.5	95.5 95.5	18.5	21.0 21.1	121 356	37 97	01	28	HILLSIDE BORROW	
7	7	CH	55	15	40	98.5	93.7 93.7	25.5	26.3 26.2	134 359	58 102	30	20	HILLSIDE BORROW	
8	8	CL	45	15	30	103.5	99.0 98.9 98.1	19.5	22.0 22.1 22.1	129 360 594	50 104 163	20	24	DOWNSTREAM VALLEY	
9	9	CL	43	17	26	101.5	96.8 96.7 96.4	21.0	23.6 23.8 24.2	132 359 579	55 103 137	40	18	VALLEY	
9A	9	CL	43	17	26	101.5	98.1 100.5 96.4	21.0	20.9 20.7 21.1	115 360 594	26 104 164	00	28	VALLEY	
9B	9	CL	43	17	26	101.5	96.7 96.6 96.6	21.0	18.9 19.1 18.7	121 343 708	36 74 186	10	26	VALLEY	
11	11	CH	51	16	35	103.5	98.9 98.9 99.0	19.5	22.1 21.8 22.0	68 247 687	31 81 150	20	20	VALLEY	
11A	11	CH	51	16	35	103.5	99.1 98.6 98.6	19.5	19.2 19.3 19.5	67 239 710	29 68 191	20	25	VALLEY	
11B	11	CH	51	16	35	103.5	98.5 98.1 97.9	19.5	17.6 17.9 18.0	64 233 702	25 57 176	10	24	VALLEY	
13	13	CH	50	17	33	103.5	98.5 99.5 98.8	19.5	22.5 22.5 22.4	124 241 578	40 70 135	18	20	OUTLET WORKS	
15	15	CH	55	16	39	102.0	93.1 93.4	20.0	23.2 23.4	122 346	38 80	19	19	SPILLWAY	
15A	15	CH	55	16	39	102.0	81.7 82.1	20.0	24.4 23.7	117 346	29 80	05	22	SPILLWAY	

\* COMPOSITE NO 6 TAKEN OUTSIDE THE PROPOSED BORROW AREA



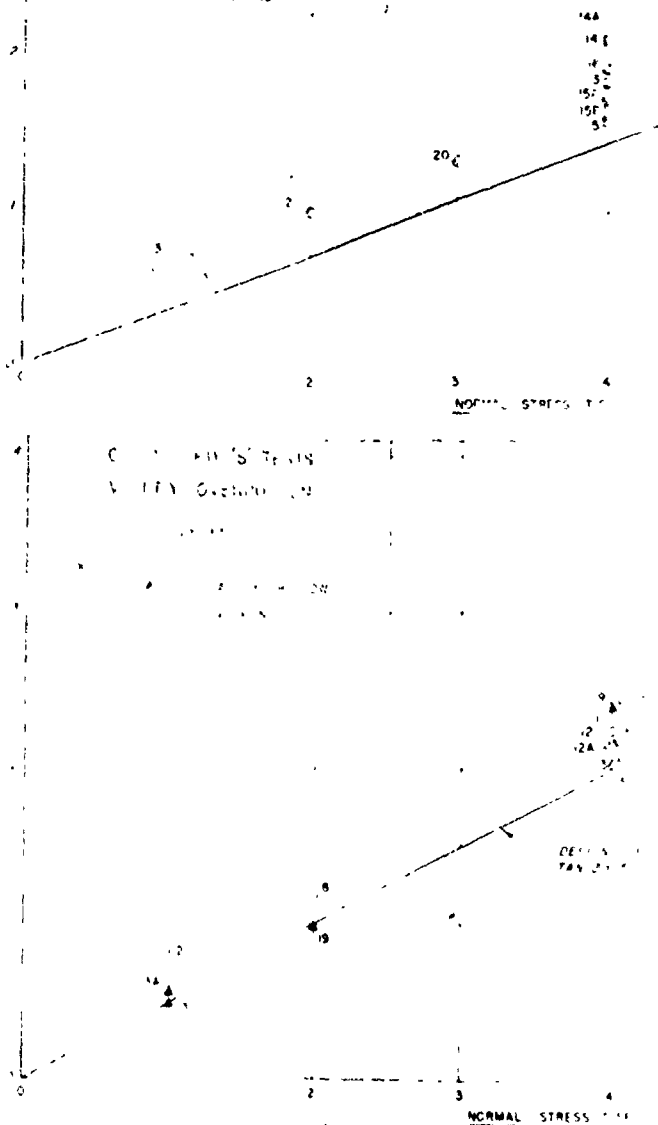


S TEST DATA ON COMPOSITE SAMPLES													
INDEX NO	COMPOSITE NO	CLASSIFICATION			DRY DENSITY		WATER CONTENT		T M <sub>50</sub>	C	TAN $\phi$	LOC.	REMARKS
		SYMBOL	LL	PL	PI	AT TEST	OPTIMUM	AT TEST					
2	2	CL	37	21	16	106.5	100.5	17.0	1	0.24	0.05	15	1A, 1B
3	3	CH	67	21	46	101.0	96.5	21.0	1	0.29	0.32	44	1A, 1B
8	8	CH	50	22	28	103.5	98.5	19.5	2	1.84	0.0	67	DOCK, 1A, 1B
9	9	CL	48	24	24	101.5	96.5	21.0	4	2.70	0.0	52	1A, 1B
10	10	CL	40	21	19	105.9	99.9	17.0	5	1.1	0.0	14	1A, 1B
11	11	CL	46	20	16	103.5	99.9	19.5	6	1.4	0.0	1	1A, 1B
12	12	CL	47	21	16	105.0	100.0	19.0	7	1.1	0.0	1	1A, 1B
12A	12	CL	47	21	16	105.0	100.0	19.0	8	1.1	0.0	1	1A, 1B
12B	12	CL	47	21	16	105.0	100.0	19.0	9	1.1	0.0	1	1A, 1B
12C	12	CL	48	16	14	105.0	94.0	19.0	10	1.1	0.0	1	1A, 1B
13	13	CH	54	22	12	105.0	98.0	19.0	11	1.1	0.0	1	1A, 1B
13A	13	CH	54	22	12	105.0	98.0	19.0	12	1.1	0.0	1	1A, 1B
13B	13	CH	54	22	12	105.0	98.0	19.0	13	1.1	0.0	1	1A, 1B
14	14	CH	53	20	10	105.0	98.0	19.0	14	1.1	0.0	1	1A, 1B
14A	14	CH	53	20	10	105.0	98.0	19.0	15	1.1	0.0	1	1A, 1B
15	15	CH	50	20	10	105.0	98.0	19.0	16	1.1	0.0	1	1A, 1B
16	16	CH	54	22	12	105.0	98.0	19.0	17	1.1	0.0	1	1A, 1B
17	17	CH	54	22	12	105.0	98.0	19.0	18	1.1	0.0	1	1A, 1B
18	18	CH	54	22	12	105.0	98.0	19.0	19	1.1	0.0	1	1A, 1B
19	19	CH	53	20	10	105.0	98.0	19.0	20	1.1	0.0	1	1A, 1B
20	20	CH	54	22	12	105.0	98.0	19.0	21	1.1	0.0	1	1A, 1B
21	21	CH	54	22	12	105.0	98.0	19.0	22	1.1	0.0	1	1A, 1B
22	22	CH	54	22	12	105.0	98.0	19.0	23	1.1	0.0	1	1A, 1B
23	23	CH	54	22	12	105.0	98.0	19.0	24	1.1	0.0	1	1A, 1B
24	24	CH	54	22	12	105.0	98.0	19.0	25	1.1	0.0	1	1A, 1B
25	25	CH	54	22	12	105.0	98.0	19.0	26	1.1	0.0	1	1A, 1B
26	26	CH	54	22	12	105.0	98.0	19.0	27	1.1	0.0	1	1A, 1B
27	27	CH	54	22	12	105.0	98.0	19.0	28	1.1	0.0	1	1A, 1B
28	28	CH	54	22	12	105.0	98.0	19.0	29	1.1	0.0	1	1A, 1B
29	29	CH	54	22	12	105.0	98.0	19.0	30	1.1	0.0	1	1A, 1B
30	30	CH	54	22	12	105.0	98.0	19.0	31	1.1	0.0	1	1A, 1B
31	31	CH	54	22	12	105.0	98.0	19.0	32	1.1	0.0	1	1A, 1B
32	32	CH	54	22	12	105.0	98.0	19.0	33	1.1	0.0	1	1A, 1B
33	33	CH	54	22	12	105.0	98.0	19.0	34	1.1	0.0	1	1A, 1B
34	34	CH	54	22	12	105.0	98.0	19.0	35	1.1	0.0	1	1A, 1B
35	35	CH	54	22	12	105.0	98.0	19.0	36	1.1	0.0	1	1A, 1B
36	36	CH	54	22	12	105.0	98.0	19.0	37	1.1	0.0	1	1A, 1B
37	37	CH	54	22	12	105.0	98.0	19.0	38	1.1	0.0	1	1A, 1B
38	38	CH	54	22	12	105.0	98.0	19.0	39	1.1	0.0	1	1A, 1B
39	39	CH	54	22	12	105.0	98.0	19.0	40	1.1	0.0	1	1A, 1B
40	40	CH	54	22	12	105.0	98.0	19.0	41	1.1	0.0	1	1A, 1B
41	41	CH	54	22	12	105.0	98.0	19.0	42	1.1	0.0	1	1A, 1B
42	42	CH	54	22	12	105.0	98.0	19.0	43	1.1	0.0	1	1A, 1B
43	43	CH	54	22	12	105.0	98.0	19.0	44	1.1	0.0	1	1A, 1B
44	44	CH	54	22	12	105.0	98.0	19.0	45	1.1	0.0	1	1A, 1B
45	45	CH	54	22	12	105.0	98.0	19.0	46	1.1	0.0	1	1A, 1B
46	46	CH	54	22	12	105.0	98.0	19.0	47	1.1	0.0	1	1A, 1B
47	47	CH	54	22	12	105.0	98.0	19.0	48	1.1	0.0	1	1A, 1B
48	48	CH	54	22	12	105.0	98.0	19.0	49	1.1	0.0	1	1A, 1B
49	49	CH	54	22	12	105.0	98.0	19.0	50	1.1	0.0	1	1A, 1B
50	50	CH	54	22	12	105.0	98.0	19.0	51	1.1	0.0	1	1A, 1B
51	51	CH	54	22	12	105.0	98.0	19.0	52	1.1	0.0	1	1A, 1B
52	52	CH	54	22	12	105.0	98.0	19.0	53	1.1	0.0	1	1A, 1B
53	53	CH	54	22	12	105.0	98.0	19.0	54	1.1	0.0	1	1A, 1B
54	54	CH	54	22	12	105.0	98.0	19.0	55	1.1	0.0	1	1A, 1B
55	55	CH	54	22	12	105.0	98.0	19.0	56	1.1	0.0	1	1A, 1B
56	56	CH	54	22	12	105.0	98.0	19.0	57	1.1	0.0	1	1A, 1B
57	57	CH	54	22	12	105.0	98.0	19.0	58	1.1	0.0	1	1A, 1B
58	58	CH	54	22	12	105.0	98.0	19.0	59	1.1	0.0	1	1A, 1B
59	59	CH	54	22	12	105.0	98.0	19.0	60	1.1	0.0	1	1A, 1B
60	60	CH	54	22	12	105.0	98.0	19.0	61	1.1	0.0	1	1A, 1B
61	61	CH	54	22	12	105.0	98.0	19.0	62	1.1	0.0	1	1A, 1B
62	62	CH	54	22	12	105.0	98.0	19.0	63	1.1	0.0	1	1A, 1B
63	63	CH	54	22	12	105.0	98.0	19.0	64	1.1	0.0	1	1A, 1B
64	64	CH	54	22	12	105.0	98.0	19.0	65	1.1	0.0	1	1A, 1B
65	65	CH	54	22	12	105.0	98.0	19.0	66	1.1	0.0	1	1A, 1B
66	66	CH	54	22	12	105.0	98.0	19.0	67	1.1	0.0	1	1A, 1B
67	67	CH	54	22	12	105.0	98.0	19.0	68	1.1	0.0	1	1A, 1B
68	68	CH	54	22	12	105.0	98.0	19.0	69	1.1	0.0	1	1A, 1B
69	69	CH	54	22	12	105.0	98.0	19.0	70	1.1	0.0	1	1A, 1B
70	70	CH	54	22	12	105.0	98.0	19.0	71	1.1	0.0	1	1A, 1B
71	71	CH	54	22	12	105.0	98.0	19.0	72	1.1	0.0	1	1A, 1B
72	72	CH	54	22	12	105.0	98.0	19.0	73	1.1	0.0	1	1A, 1B
73	73	CH	54	22	12	105.0	98.0	19.0	74	1.1	0.0	1	1A, 1B
74	74	CH	54	22	12	105.0	98.0	19.0	75	1.1	0.0	1	1A, 1B
75	75	CH	54	22	12	105.0	98.0	19.0	76	1.1	0.0	1	1A, 1B
76	76	CH	54	22	12	105.0	98.0	19.0	77	1.1	0.0	1	1A, 1B
77	77	CH	54	22	12	105.0	98.0	19.0	78	1.1	0.0	1	1A, 1B
78	78	CH	54	22	12	105.0	98.0	19.0	79	1.1	0.0	1	1A, 1B
79	79	CH	54	22	12	105.0	98.0	19.0	80	1.1	0.0	1	1A, 1B
80	80	CH	54	22	12	105.0	98.0	19.0	81	1.1	0.0	1	1A, 1B
81	81	CH	54	22	12	105.0	98.0	19.0	82	1.1	0.0	1	1A, 1B
82	82	CH	54	22	12	105.0	98.0	19.0	83	1.1	0.0	1	1A, 1B
83	83	CH	54	22	12	105.0	98.0	19.0	84	1.1	0.0	1	1A, 1B
84	84	CH	54	22	12	105.0	98.0	19.0	85	1.1	0.0	1	1A, 1B
85	85	CH	54	22	12	105.0	98.0	19.0	86	1.1	0.0	1	1A, 1B
86	86	CH	54	22	12	105.0	98.0	19.0	87	1.1	0.0	1	1A, 1B
87	87	CH	54	22	12	105.0	98.0	19.0	88	1.1	0.0	1	1A, 1B
88	88	CH	54	22	12	105.0	98.0	19.0	89	1.1	0.0	1	1A, 1B
89	89	CH	54	22	12	105.0	98.0	19.0	90	1.1	0.0	1	1A, 1B
90	90	CH	54	22	12	105.0	98.0	19.0	91	1.1	0.0	1	1A, 1B
91	91	CH	54	22	12	105.0	98.0	19.0	92	1.1	0.0	1	1A, 1B
92	92	CH	54	22	12	105.0	98.0	19.0	93	1.1	0.0	1	1A, 1B
93	93	CH	54	22	12	105.0	98.0	19.0	94	1.1	0.0	1	1A, 1B
94	94	CH	54	22	12	105.0	98.0	19.0	95	1.1	0.0	1	1A, 1B
95	95	CH	54	22	12	105.0	98.0	19.0	96	1.1	0.0	1	1A, 1B
96	96	CH	54	22	12	105.0	98.0	19.0	97	1.1	0.0	1	1A, 1B
97	97	CH	54	22	12	105.0	98.0	19.0	98	1.1	0.0	1	1A, 1B
98	98	CH	54	22	12	105.0	98.0	19.0	99	1.1	0.0	1	1A, 1B
99	99	CH	54	22	12	105.0	98.0	19.0	100	1.1	0.0	1	1A, 1B

# COMPACTED "S" TESTS HILLSIDE OVERBURDEN

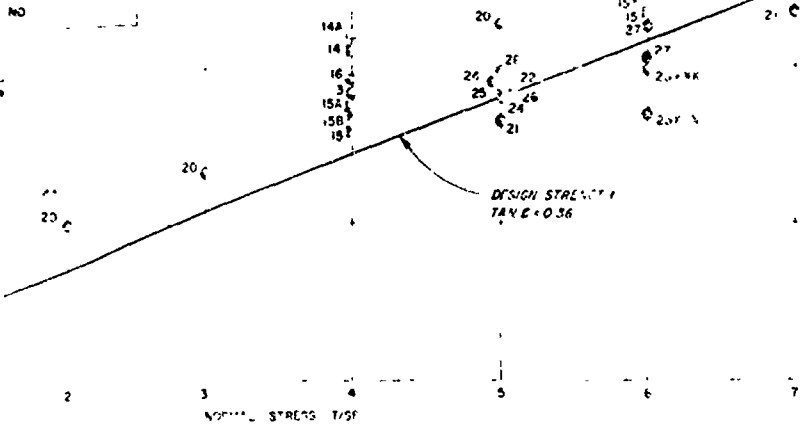
## LEGEND

- CL CH
- HILLSIDE BORROW
- SAILWAY BORROW
- HILLSIDE BORROW
- OUTSIDE PROPOSED AREA
- NCX NC



# TESTS

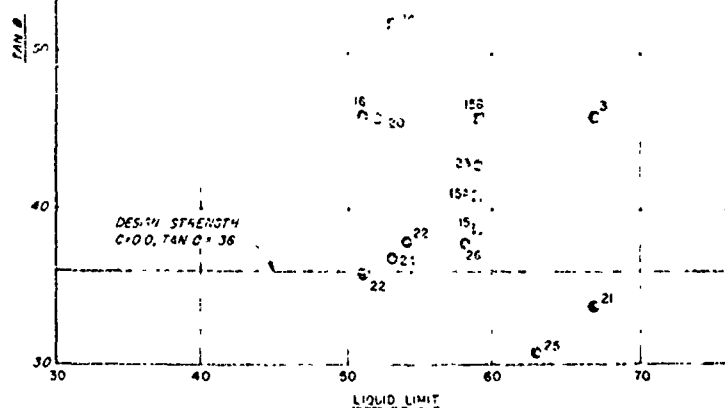
DE BORROW  
BY BORROW  
E BORROW  
E PROPOSED



## COMPACTED "S" TESTS HILLSIDE OVERBURDEN

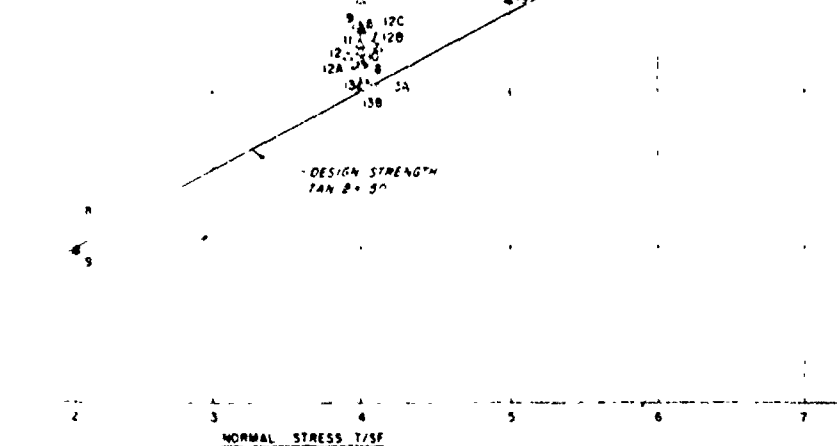
### LEGEND

SL CH  
C HILLSIDE BORROW  
F SPILLWAY BORROW  
C HILLSIDE BORROW  
(OUTSIDE PROPOSED AREA)  
S INDEX NO



# TESTS

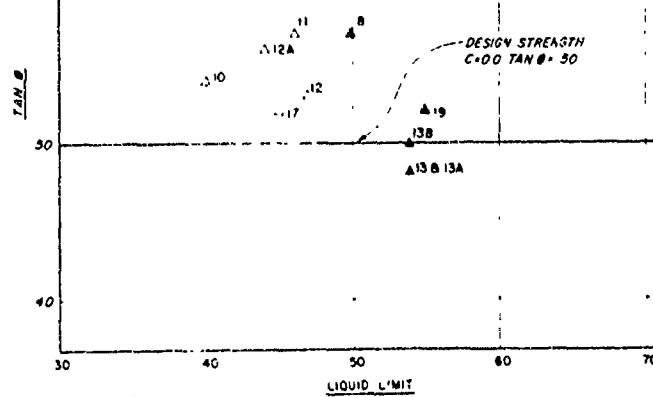
DE BORROW  
BY BORROW  
E BORROW  
E PROPOSED



## COMPACTED "S" TESTS VALLEY OVERBURDEN

### LEGEND

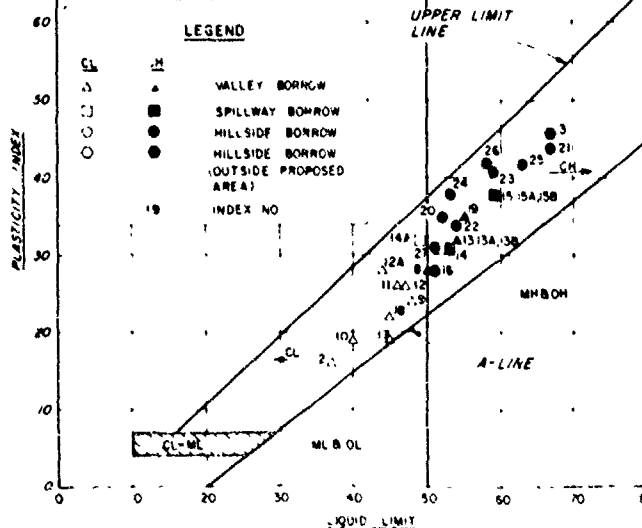
SL CH  
VALLEY BORROW  
S INDEX NO



## ATTERBERG LIMITS SHEAR TESTS

### LEGEND

SL CH  
VALLEY BORROW  
SPILLWAY BORROW  
HILLSIDE BORROW  
HILLSIDE BORROW  
(OUTSIDE PROPOSED AREA)  
S INDEX NO



SYM

DESCRIPTION  
REVISIONS

DATE APPD

LITTLE BLUE RIVER, MISSOURI

## LONGVIEW LAKE

EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
TEST DATA SUMMARY  
COMPACTED EMBANKMENT MATERIAL  
S-TESTS AND ATTERBERG LIMIT TESTS

In 1 sheet  
CORPS OF ENGINEERS  
KANSAS CITY DISTRICT

Sheet No. 1

Scale as shown  
U. S. ARMY  
JUNE 1978

Submitted  
R.G.D.

Recommended  
R.G.D.

Approved  
R.G.D.

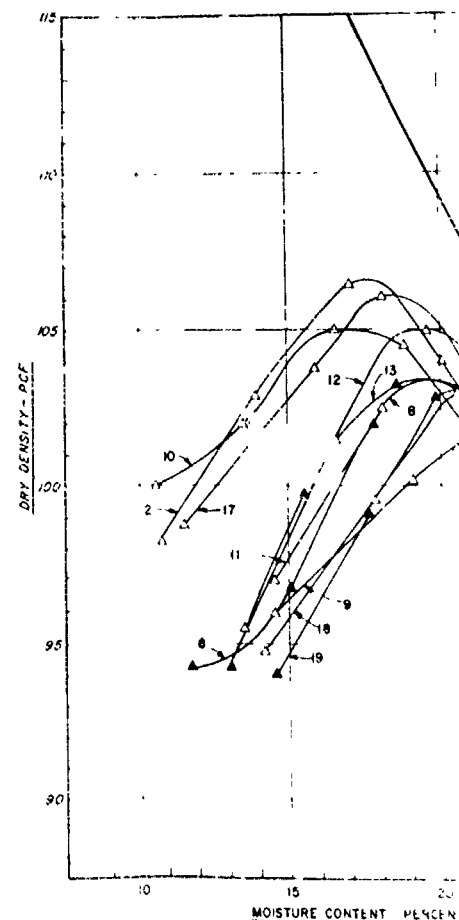
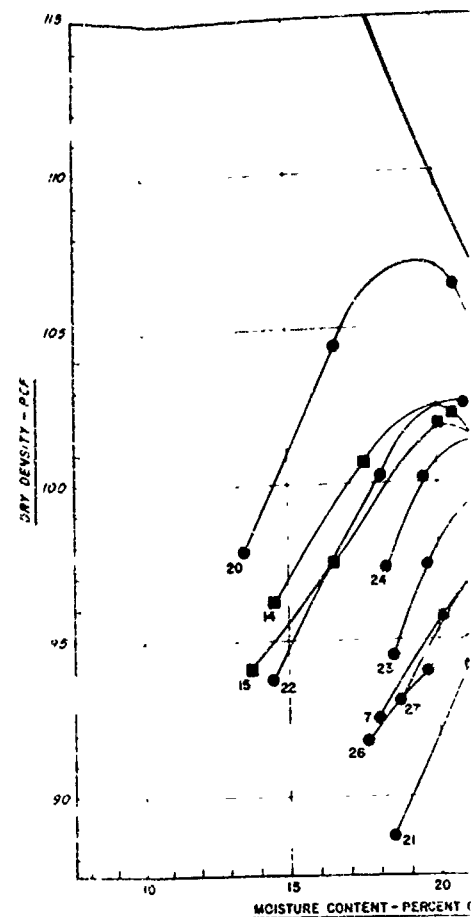
RBL-3-1688

PLATE NO. 86C

DATA ON COMPOSITE SAMPLES										
INDEX NUMBER	COMP NUMBER	HOLE NUMBER	SACK NUMBER	DEPTH	CLASSIFICATION			MAXIMUM DRY DENSITY	OPTIMUM WATER CONTENT	LOCATION
					SYMBOL	LL	PL			
2	2	A 33	1.62	103.0	CL	37	21	106.5	17.0	VALLEY BORROW
		A 34	2	60.110						
		P 76	1	05.78						
3	3	A 75	1	05.75	CH	67	21	101.0	21.0	HILLSIDE BORROW
		A 76	1	05.0						
4	4	A 79	1.62	05.0	CH	55	24	102.0	19.5	HILLSIDE BORROW
		A 79	1.62	05.0						
		A 79	1.62	05.0	CH	55	24	112.0	16.0	HILLSIDE BORROW
		A 79	1.62	05.0						
		A 66	1	30.125	CH	55	21	103.5	19.5	HILLSIDE BORROW
		A 69	1	06.67						
		A 68	1	30.125	CH	55	21	111.5	16.6	HILLSIDE BORROW
		A 69	1	06.67						
		A 81	1	20.02	CL	44	17	95.5	18.5	HILLSIDE BORROW
		A 67	1	25.110						
		A 72	1	015.0	CH	55	15	98.5	22.5	HILLSIDE BORROW
		A 80	1	032.0						
		D 90	1	025.0	CH	50	22	103.5	19.5	DOWNSTREAM BORROW
		D 91	1	017.8						
		D 84	1	012.0	CL	48	24	101.5	21.0	VALLEY BORROW
		D 85	1	0.8						
		A 62	1	05.5	CL	40	21	105.0	17.0	VALLEY BORROW
		A 63	1	05.8						
		A 73	1	05.4	CL	46	20	103.5	19.5	VALLEY BORROW
		D 86	1	0.75						
		A 57	1.62	10.20.0						
		AC 21	3.64	16.0.26.0						
		AC 30	1.62	2.0.0.0						
		AC 55	1	10.7.5	CH	54	27	102.5	19.5	OUTLET WORKS
		AC 52	1	10.11.0	CH	53	22	103.0	19.5	SPILLWAY
		AC 53	1	10.7.0						
		AC 52	2	11.5.15.0	CH	59	21	102.0	20.0	SPILLWAY
		AC 54	1	1.0.0.0						
		A 65	1	17.12.7	CH	51	23	102.0	20.0	HILLSIDE BORROW
		A 66	1	03.4						
		D 171	1	4.0.6.0	CL	43	24	106.0	18.0	VALLEY BORROW
		D 172	2	8.0.11.0						
		D 173	1	2.0.4.0	CL	45	23	102.5	20.5	VALLEY BORROW
		D 40A	1	2.0.4.0						
		D 20A	1.62	23.5.6.4.6.0	CH	55	20	103.0	20.5	VALLEY BORROW
		D 175	1	6.0.0.0	CH	52	17	107.0	19.0	HILLSIDE BORROW
		D 184	1	2.0.8.0						
		D 176	1	2.0.4.0	CH	67	23	96.0	23.5	HILLSIDE BORROW
		D 177	1	2.0.4.0						
		D 180	1	2.0.4.0	CH	54	20	102.5	20.5	HILLSIDE BORROW
		D 181	1	4.0.6.0						
		D 131A	2	4.0.6.0	CH	59	18	100.0	22.2	HILLSIDE BORROW
		D 184	1	2.0.4.0						
		D 131A	1	4.0.6.0	CH	53	15	107.0	21.2	HILLSIDE BORROW
		D 182	2	8.4.10.0						
		D 88	2	12.0.21.5	CH	63	21	96.5	24.3	HILLSIDE BORROW
		A 135	2	6.0.11.0						
		A 71	1	2.5.10.0	CH	58	16	96.0	23.3	HILLSIDE BORROW
		D 178	2	10.0.12.0						
		D 184	1	2.0.4.0	CH	51	20	98.5	22.8	HILLSIDE BORROW
		D 18A	2	10.0.12.0						
		D 178	2	8.0.10.0						

# MODIFIED PROCTOR TEST

NOTE COMPOSITES NO. 3, 4, 5, 6 AND 16 ARE OUTS. OF THE PROPOSED BORROW AREA

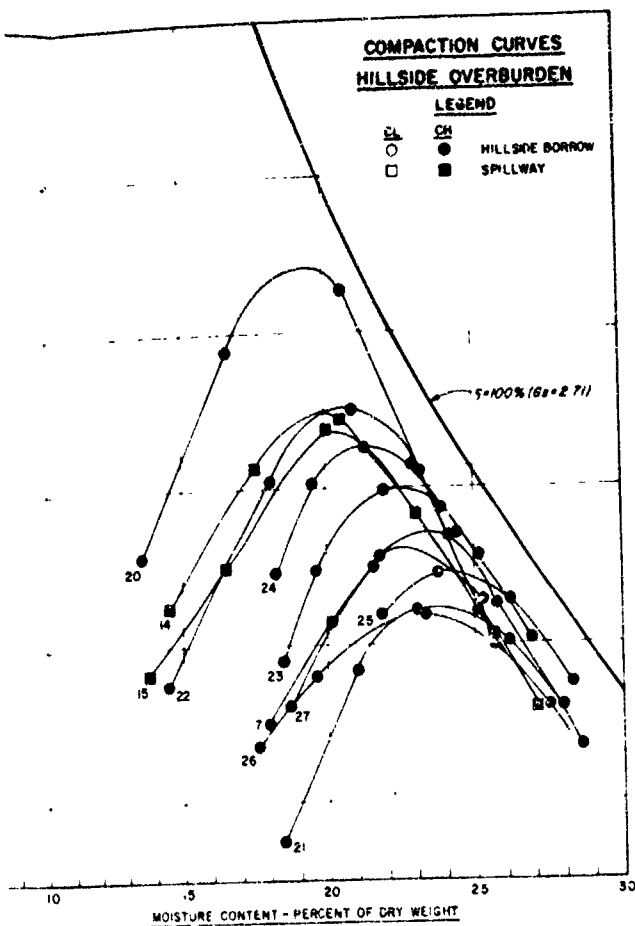


20  
PERCENT

### COMPACTION CURVES HILLSIDE OVERBURDEN

#### LEGEND

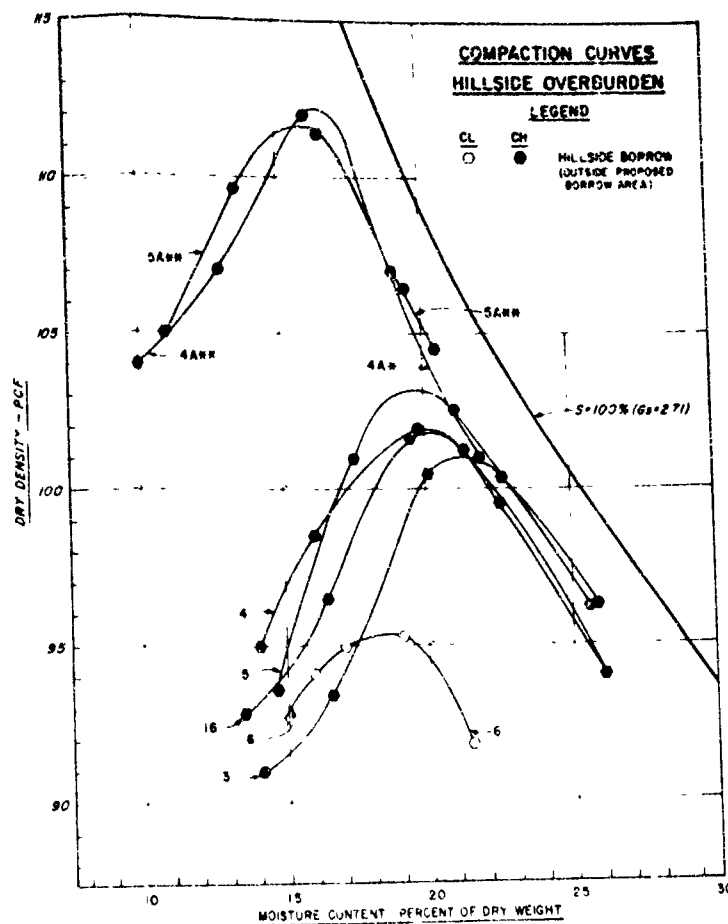
- CL CH  
○ ● HILLSIDE BORROW  
□ ■ SPILLWAY



### COMPACTION CURVES HILLSIDE OVERBURDEN

#### LEGEND

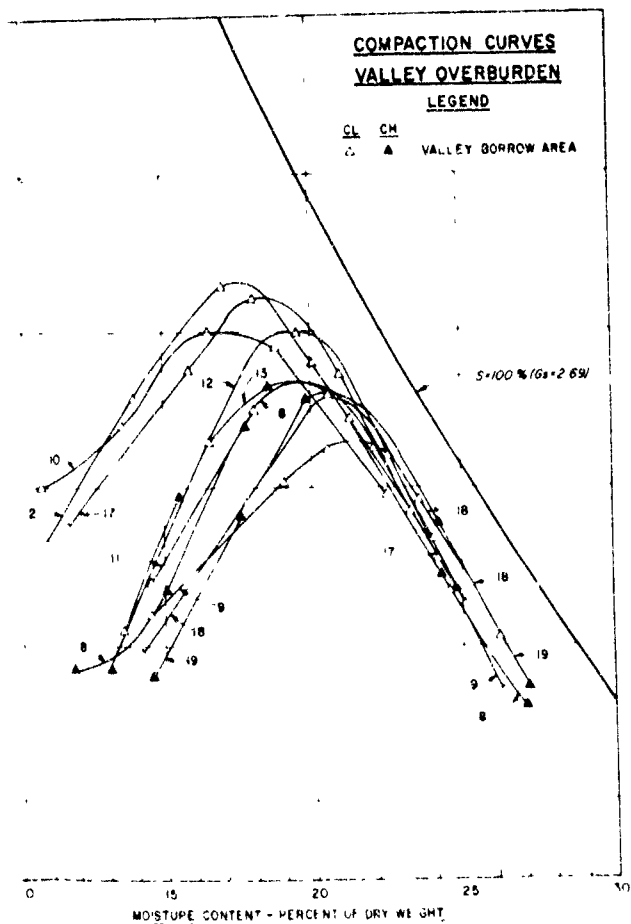
- CL CH  
○ ● HILLSIDE BORROW  
(OUTSIDE PROPOSED  
BORROW AREA)



### COMPACTION CURVES VALLEY OVERBURDEN

#### LEGEND

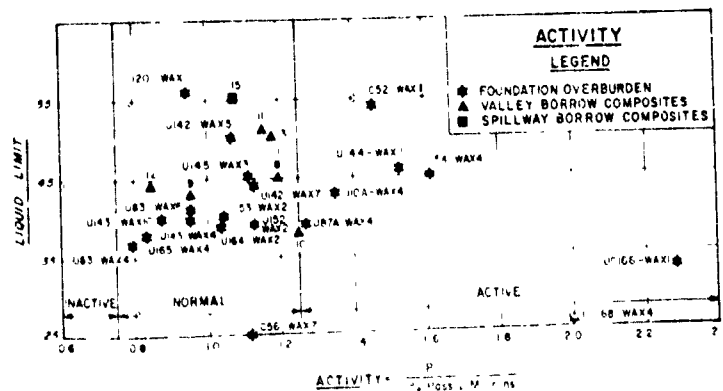
- CL CH  
○ ▲ VALLEY BORROW AREA



### ACTIVITY

#### LEGEND

- ★ FOUNDATION OVERBURDEN  
▲ VALLEY BORROW COMPOSITES  
■ SPILLWAY BORROW COMPOSITES



NOTE: Composites 4A B 1A are from Modified Proctor Test

SYM

REVISION

DATE: APR 78

LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
EMBANKMENT CRITERIA AND PERFORMANCE REPORT  
TEST DATA SUMMARY  
COMPACTION CURVES  
GRADATIONS AND ACTIVITY DATA

Prepared by  
U.S. Army Corps of Engineers  
Vicksburg District  
Vicksburg, Mississippi

Scale as shown

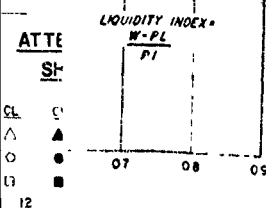
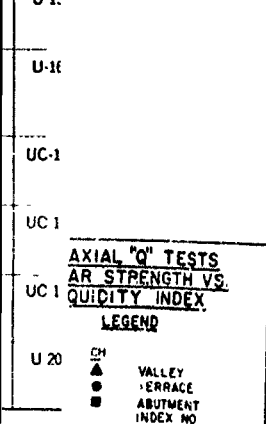
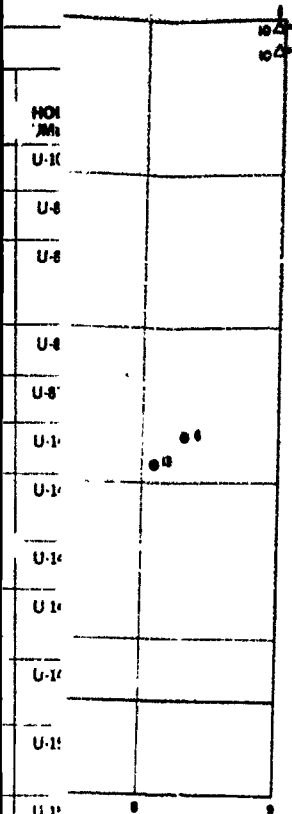
U.S. ARMY  
JUNE 1978

RBL-3-1689

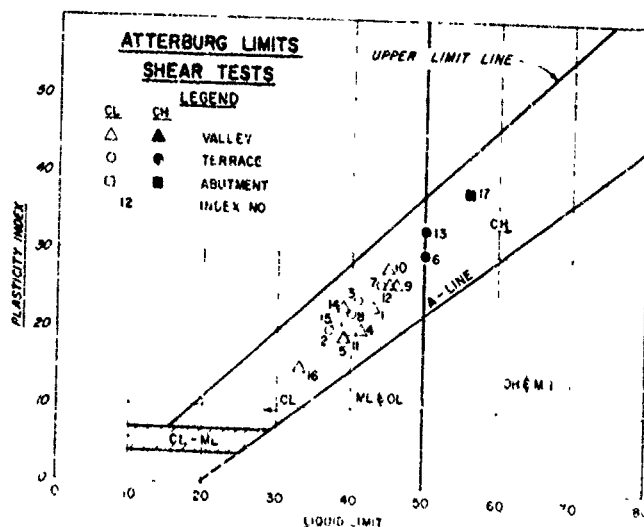
PLATE NO. 860



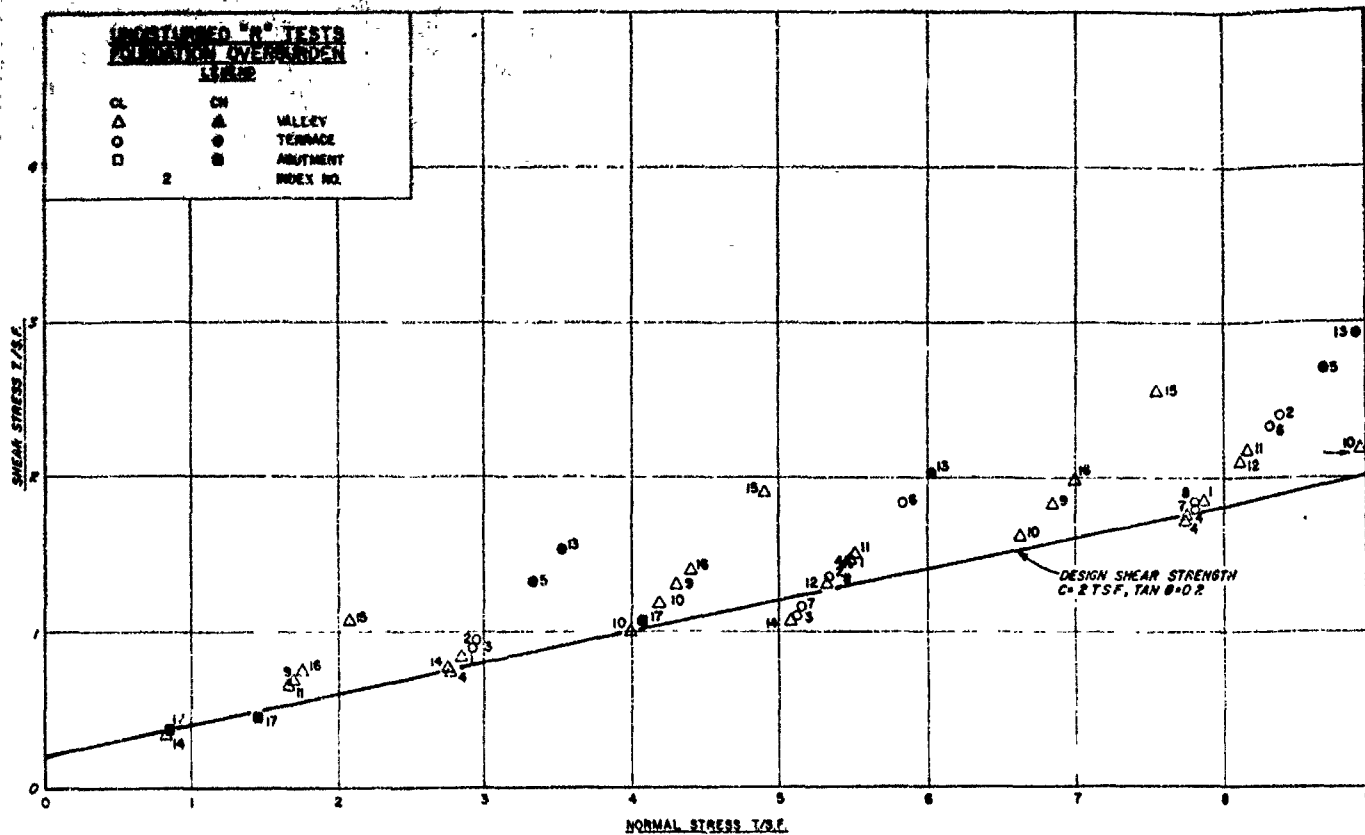




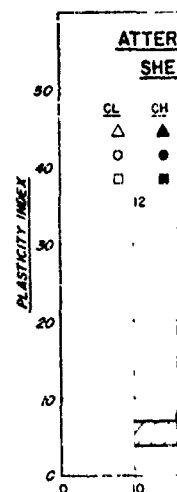
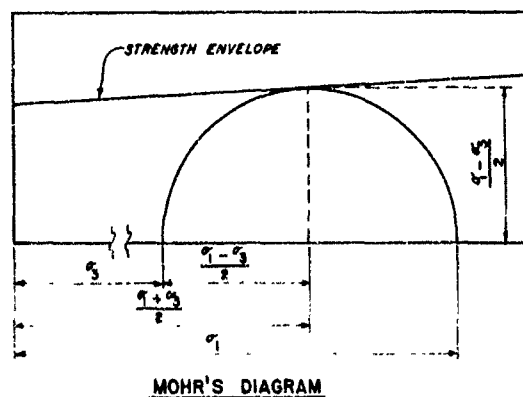
TRIAXIAL "Q" TEST DATA																
INDEX NUMBER	HOLE NUMBER	SAMPLE NUMBER	DEPTH	CLASSIFICATION			INITIAL CONDITIONS			$\sigma_3$	$\sigma_1$	$(\sigma_1 - \sigma_3)/2$	$(\sigma_1 + \sigma_3)/2$	C	TAN $\phi$	LOCATION
				SYMBOL	L. L.	P. L.	DRY DENSITY	WATER CONTENT	% SAT.							
1	U-10A	WAX 4	7.0-7.9	CL	48	20	87.6 88.1	32.3 31.6	98 98	4 8	5.85 7.13	.53 .55	4.53 6.56	0.52	0.0	VALLEY
2	U-83	WAX 4	7.2-7.9	CL	37	17	98.6 98.7	23.4 23.7	90 91	4 6	6.24 8.27	1.12 1.13	5.12 7.13	1.00	.08	TERRACE
3	U-83	WAX 6	10.0-11.2	CL	41	17	94.4 94.6 94.6 92.9	26.4 26.3 27.1 26.0	93 93 96 88	2 4 1 4	2.86 5.77 2.84 5.74	.38 .86 .52 .87	2.86 4.86 1.92 4.87	0.4	.10	TERRACE
4	U-87	WAX 3	4.0-5.9	CL	41	21	81.8 83.4	22.2 21.4	98 94	2 4	5.18 7.94	1.98 1.97	3.98 5.97	1.0	.18	VALLEY
5	U-87A	WAX 4	7.5-7.9	CL	38	20	91.2 90.8	29.8 29.7	98 97	4 6	5.08 7.05	.94 .52	4.94 6.52	.52	0.0	VALLEY
6	U-142	WAX 5	7.1-8.0	CH	30	20	97.2 98.5	24.1 24.6	88 95	2 6	5.87 10.82	1.83 2.31	3.83 8.31	.95 (CH-8) 2.4 (CH-6)	.24 0.0	TERRACE
7	U-142	WAX 7	10.0-11.5	CL	44	18	93.9 93.3 94.6	28.1 27.7 27.7	96 98 96	4 6 2	6.02 8.19 3.85	1.01 1.08 .63	5.01 7.08 2.83	.6 (CH-8) 1.1 (CH-6)	.08 0.0	TERRACE
8	U-143	WAX 10	18.0-19.0	CL	40	18	96.2 95.9	28.1 29.5	100 100	4 6	5.05 6.96	.52 .46	4.52 6.46	.50	0.0	TERRACE
9	U-144	WAX 1	0.5-1.8	CL	46	20	88.6 88.9 89.0	26.9 26.6 27.1	83 84 84	1 3 5	3.22 6.21 8.80	1.11 1.87 1.80	2.11 4.81 6.80	0.8 (CH-6) 1.8 (CH-6)	.18 0.0	VALLEY
10	U-145	WAX 5	4.0-5.9	CL	45	17	93.5 95.2 93.2	16.3 16.1 16.4	56 57 63	3 5 7	11.29 14.80 18.18	4.14 4.80 5.99	7.14 9.80 12.59	2.4	.28	VALLEY
11	U-153	WAX 2	2.0-3.6	CL	40	20	89.6 89.6 91.6	28.7 28.0 27.7	90 88 91	1 4 6	2.45 6.00 7.92	.72 1.00 .96	1.72 5.00 6.96	.75	0.0	VALLEY
12	U-154	WAX 4	6.0-7.9	CL	45	'9	90.8 87.7 88.1	24.7 24.9 23.7	73 73 72	4 6 2	6.24 8.56 4.36	1.12 1.28 1.18	5.12 7.28 3.18	1.2	0.0	VALLEY
13	U-163	WAX 3	4.0-5.9	CH	50	17	100.7 101.3 101.2 101.5	22.7 21.9 22.1 22.6	90 86 88 92	2 4 6 ?	4.83 8.43 10.28 5.00	1.41 2.21 2.14 1.50	3.41 6.21 8.14 3.30	0.7 (CH-8) 2.1 (CH-6)	.24 0.0	TERRACE
14	UC-164	WAX 2	2.0-3.9	CL	38	16	97.4 97.0 96.9	20.6 20.6 20.9	77 76 77	5 2 4	2.08 4.62 7.11	.79 1.31 1.55	1.29 3.31 5.55	0.5 (CH-3) 1.0 (CH-3)	.23 .10	VALLEY
15	UC-185	WAX 4	6.0-7.9	CL	38	17	100.6 101.0 101.1	22.8 23.4 23.4	94 97 97	1 3 5	2.75 4.77 6.93	.87 .88 .96	1.87 3.88 5.96	.85	.02	VALLEY
16	UC-186	WAX 3	4.0-5.7	CL	33	18	100.5 100.6 100.0	24.2 24.3 24.2	98 99 97	1 3 5	2.56 4.39 6.39	.78 .69 .69	1.78 3.69 5.69	.70	0.0	VALLEY
17	U 201	WAX 1	1.0-2.0	CH	56	18	96.6 96.9 96.7	26.2 26.2 26.1	97 98 97	5 1 3	2.11 2.74 4.84	.80 .87 .92	1.30 1.87 3.92	.75	.05	ABUTMENT



SYM	DESCRIPTION	DATE	APP'D
	REVISIONS		
	LITTLE BLUE RIVER, MISSOURI		
	<b>LONGVIEW LAKE</b>		
	EMBANKMENT CRITERIA AND		
	PERFORMANCE REPORT		
	TEST DATA SUMMARY		
	UNDISTURBED FOUNDATION OVERBURDEN		
	Q-TESTS		
In 1 sheet	Sheet No. 1	Scale as shown	
CORPS OF ENGINEERS		U S ARMY	
KANSAS CITY DISTRICT		JUNE 1978	
Submitted	Recommended	Approved	
Checked by	Checked by	Checked by	
GLG	JPM	RGD	
			RBL-3-1690



INDEX NUMBER	HOLE NUMBER
1	U-10A
2	U-83
3	U-83
4	U-87
5	U-142
6	U-142
7	U-143
8	U-143
9	U-144
10	U-145
11	U-153
12	U-154
13	U-163
14	UC-164
15	UC-165
16	UC-166
17	U-201



OLE

MBCR

-83

-83

-87

-142

-142

-143

-143

-144

-145

-153

U-154

U-163

U-164

U-165

U-166

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

U-201

## TRIAXIAL "R" TEST DATA

INDEX NUMBER	HOLE NUMBER	SAMPLE NUMBER	DEPTH	CLASSIFICATION			INITIAL CONDITIONS			$\sigma_3$	$\sigma_1$	$\frac{\sigma_1 - \sigma_3}{2}$	$\frac{\sigma_1 + \sigma_3}{2}$	C	TAN $\phi$	LOCATION
				SYMBOL	LL	PL	DRY DENSITY	WATER CONTENT	% SAT.							
1	U-10A	WAX 4	7.0-7.9	CL	43	20	81.1	33.7	86	2	3.89	.84	2.84	.25	.22	VALLEY
							85.8	32.5	92	4	6.92	1.46	5.46			
							85.1	32.3	90	6	9.71	1.86	7.86			
2	U-83	WAX 4	7.2-7.9	CL	37	17	86.7	23.6	91	4	6.68	1.34	5.34	.20	.26	TERRACE
							86.4	23.8	92	6	10.79	2.40	8.40			
							86.9	23.8	92	2	3.80	.96	2.86			
3	U-83	WAX 6	10.0-11.9	CL	41	17	84.5	26.2	83	2	3.82	.91	2.91	0.70	.33	TERRACE
							85.2	26.3	97	4	6.25	1.12	5.12			
4	U-87	WAX 3	4.0-5.9	CL	41	21	87.3	22.6	88	2	3.84	.77	2.77	.25	.21	VALLEY
							87.4	23.9	72	4	6.94	1.47	5.47			
							82.0	21.1	56	6	9.49	1.74	7.74			
							84.4	20.6	58	6	9.50	1.75	7.75			
5	U-142	WAX 6	7.1-8.0	CH	50	20	86.3	25.0	83	2	4.64	1.32	3.32	.50	.27	TERRACE
							86.6	25.0	90	6	11.39	2.70	8.70			
6	U-142	WAX 7	10.0-11.9	CL	44	19	88.2	27.8	93	4	7.67	1.84	5.84	1.00	.17	TERRACE
							93.2	27.5	93	6	10.67	2.34	8.34			
7	U-143	WAX 4	6.0-7.9	CL	40	16	87.2	24.7	92	4	6.29	1.14	5.14	0.0	.24	TERRACE
							87.7	24.7	93	6	9.55	1.78	7.78			
8	U-143	WAX 10	18.0-19.0	CL	40	18	84.1	28.1	96	4	6.68	1.34	5.34	.25	.21	TERRACE
							84.0	28.8	96	6	9.66	1.83	7.83			
9	U-144	SHELBY 1	0.5-1.8	CL	46	20	91.7	26.6	88	1	2.39	.70	1.70	.30	.20	VALLEY
							91.4	26.6	88	3	5.60	1.30	4.30			
							93.2	26.5	91	5	8.67	1.84	6.84			
10	U-145	SHELBY 3	4.0-5.9	CL	45	17	87.0	15.7	58	3	5.37	1.18	4.18	.20	.23	VALLEY
							86.1	16.2	59	5	8.25	1.62	6.62			
							86.1	16.2	59	7	11.37	2.18	9.18			
							86.5	16.2	60	3	5.01	1.00	4.00			
11	U-153	SHELBY 2	2.0-3.6	CL	40	20	87.1	28.2	84	1	2.33	.68	1.68	.30	.235	VALLEY
							87.2	28.7	85	4	7.03	1.52	5.52			
							87.5	28.3	85	6	10.34	2.17	8.17			
12	U-154	WAX 4	6.0-7.9	CL	45	19	88.4	27.7	83	4	6.63	1.32	5.32	0.0	.26	VALLEY
							90.8	27.1	86	6	10.23	2.12	8.12			
13	U-163	SHELBY 3	4.0-5.9	CH	50	17	102.2	21.8	90	2	5.09	1.54	3.54	.90	.22	TERRACE
							102.3	21.8	90	4	8.05	2.02	6.02			
							102.5	22.4	93	6	11.84	2.92	8.92			
14	UC-164	WAX 2	2.0-3.9	CL	38	16	85.0	21.3	75	0.5	1.19	.34	.84	.25	.17	VALLEY
							86.7	21.7	80	2	3.53	.76	2.76			
							86.0	21.6	78	4	6.18	1.08	5.08			
15	UC-165	SHELBY 4	6.0-7.9	CL	38	17	101.5	23.4	98	1	3.17	1.08	2.08	.60	.38	VALLEY
							101.2	23.4	98	3	6.76	1.88	4.88			
							101.6	23.3	98	5	10.09	2.54	7.54			
16	UC-166	SHELBY 3	4.0-5.7	CL	33	18	99.9	24.5	98	1	2.51	.76	1.76	.40	.24	VALLEY
							100.8	24.7	100+	3	5.77	1.38	4.38			
							101.2	24.6	100+	5	8.96	1.98	6.98			
17	U-201	SAMPLE 1	1.0-2.0	CH	56	18	84.9	28.1	100	0.5	1.22	.36	.86	.20	.22	ABUTMENT
							85.7	28.1	100+	1	1.90	.45	1.45			
							93.7	27.8	96	3	5.14	1.07	4.07			

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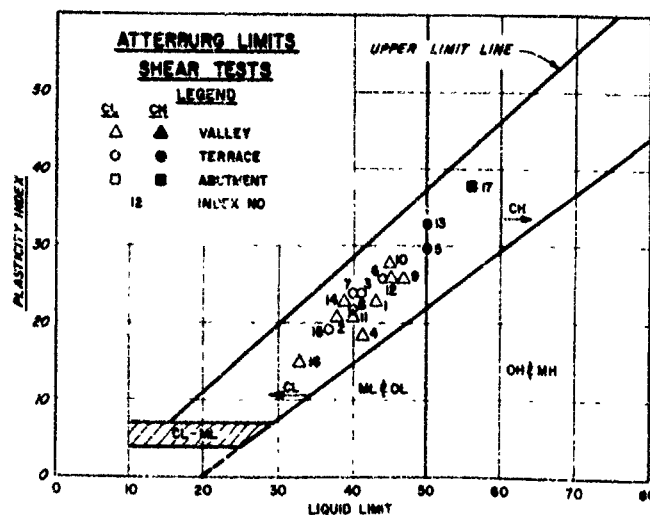
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SYM

DESCRIPTION  
REVISIONS

DATE APP'D

LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
 EMBANKMENT CRITERIA AND  
 PERFORMANCE REPORT  
 TEST DATA SUMMARY  
 UNDISTURBED FOUNDATION OVERBURDEN  
 R-TESTS

In 1 sheet  
 CORPS OF ENGINEERS  
 KANSAS CITY DISTRICT

Sheet No 1

Scale as shown  
 U S ARMY  
 JUNE 1978

Submitted  
 Checked by  
 G L S O H

Recommended  
 Chief Foundations & Mats  
 R G D

Approved  
 Chief Engineering Division  
 RBL-3-1691

PLATE NO. 86F

# SUMMARY UNDISTURBED "S" TEST DATA OVERBURDEN MATERIALS

INDEX NUMBER	HOLE NUMBER	SAMPLE NUMBER	DEPTH	CLASSIFICATION				INITIAL CONDITIONS			0% T.S.F.	T MAX. T.S.F.	RESIDUAL T.S.F.	C T.S.F.	TAN $\phi$	LOCATION
				SYMBOL	L. L.	P. L.	P. I.	DRY DENSITY	WATER CONTENT	% SAT.						
1	UC-9	WAX 2	33.37	OH	51	30	21	80.5	34	85	6	3.8 3.9		0.0	0.60	VALLEY
2	UC-9	WAX 4	68.72	CL	50	23	27	91.5	30.5	98	6	4.0 3.76		0.0	0.65	VALLEY
3	U-9A	WAX 4	74.76	CL	42	21	21	91.5	27	89	2	1.02 1.31		0.0	0.58	VALLEY
4	U-10A	WAX 4	65.68	CL	46	24	22	91.0	31.5	100	6	3.81 3.60		0.0	0.62	VALLEY
5	U-10B	WAX 3	49.53	CL	44	24	20	89.0	25.0	77	4	2.41 2.65		0.0	0.63	VALLEY
6	U-10B	WAX 5	85.90	CL	41	21	20	95.0	33.0	95	4	2.20 2.75		0.0	0.67	VALLEY
7	U-82	WAX 1	10.15	CL	44	27	7	91.5	28	93	6	4.24 3.84		0.0	0.67	VALLEY
8	U-82	WAX 5	88.91	CL	40	22	18	97.0	26	97	6	3.74 3.84		0.0	0.63	VALLEY
9	U-83	WAX 2	28.31	CL	34	23	11	90.5	14.5	90.5	4	2.70 3.81		0.5	0.74	TERRACE
10	U-83	WAX 2	28.31	CL	34	23	11	90.5	14.5	90.5	4		2.50 3.50	0.5	0.70	TERRACE
11	U-83	WAX 4	66.69	CL	42	22	20	94.5	24.0	84	4	2.59 3.57		0.5	0.53	TERRACE
12	U-83	WAX 6	100.11.9	CL	41	17	24	92.1 91.0 95.7	27.9 28.2 28.2	93 92 100	2 4 6	1.06 2.24 3.58		0.0	0.53	TERRACE
13	U-83	WAX 10	18.618.9	CL	38	23	15	96.0	27.5	98	4	2.39 3.62		0.0	0.69	TERRACE
14	U-87	WAX 1	10.13	CL	40	24	16	87.5	20.0	87.5	6	3.35 3.92		0.0	0.62	VALLEY
15	U-87	WAX 1	10.13	CL	40	24	16	87.5	20.0	87.5	6		3.40 3.69	0.0	0.72	VALLEY
16	U-87	WAX 3	40.5.9	CL	41	21	20	79.3 81.9 87.8	21.0 19.0 17.9	52 50 55	2 4 6	1.42 2.63 4.37		0.9	0.7	VALLEY
17*	U-87	WAX 3	40.5.9	CL	41	21	20	87.3 87.4	22.6 23.9	68 72	1 2.2	0.65 1.40		0.0	0.65	VALLEY
18	U-87	WAX 4	61.6.6	CL	46	22	24	90.0	27.5	90	6	3.57 3.74		0.0	0.61	VALLEY
19	U-87A	WAX 1	12.1.6	CL	46	24	22	92.0	28.0	92	5	3.04 3.28		0.0	0.65	VALLEY
20	U-87A	WAX 4	69.7.3	CL	46	24	22	85.0	31.0	85.5	6	3.60 3.93		0.0	0.63	VALLEY
21	U-142	WAX 3	53.5.9	CH	61	19	42	97.0	25.5	93	2 4 6 6	1.14 2.25 2.90 2.94		0.16	0.46	TERRACE
22	U-142	WAX 4	65.6.8	CH	57	24	33	95.5	26.0	93	2 4 6 6	1.10 2.29 3.09		0.22	0.49	TERRACE
23	U-142	WAX 5	80.8.9	CH	52	26	26	92.5	25.5	92.5	6	2.74 3.18		0.0	0.49	TERRACE
24	U-142	WAX 5	80.8.9	CH	52	26	26	92.5	25.5	92.5	6		2.40 3.06	0.0	0.46	TERRACE
25	U-142	WAX 9	15.415.9	CL	44	19	25	96.5	27.0	99	4 6	2.31 3.59		0.0	0.60	TERRACE
26	U-143	WAX 4	60.9.0	CL	40	16	24	96.2	24.6	89	6	3.36		0.0	0.56	TERRACE
27	U-143	WAX 4	60.7.9	CL	40	16	24	96.2 96.7	24.6 24.2	89 93	6		3.28 3.39	0.0	0.54	TERRACE
28*	U-143	WAX 4	60.7.9	CL	40	16	24	97.2 97.7	24.7 24.7	92 93	19 2.5	50 1.75		0.0	0.53	TERRACE
29	U-143	WAX 5	90.9.8	CL	42	18	24	95	27	93	2 6	1.34 3.53		0.25	0.55	TERRACE
30	U-143	WAX 9	17.017.5	CL	43	21	22	94	28.5	97	1 4 6	67 2.59 3.45		0.23	0.55	TERRACE
31	U-143	WAX 10	19.319.7	CL	44	22	27	86.5	31	87	6 6	3.45 1.04		0.0	0.625	TERRACE
32	U-143	WAX 12	22.227.7	CL	49	18	31	90.5	31.5	100	6 6	3.25 3.62		0.0	0.57	TERRACE
33*	U-144	WAX 1	0.51.8	CL	46	20	26	93.2	26.5	91	2.4	1.75		0.0	0.70	VALLEY

\* NOTE

INDEX NUMBER
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DATE \* APR 20 1964

Sheet No. 1

Scale as shown  
U S ARMY  
JUNE 1978

in sheet  
CORPS OF ENGINEERS  
KANSA, CITY DISTRICT  
Submitted

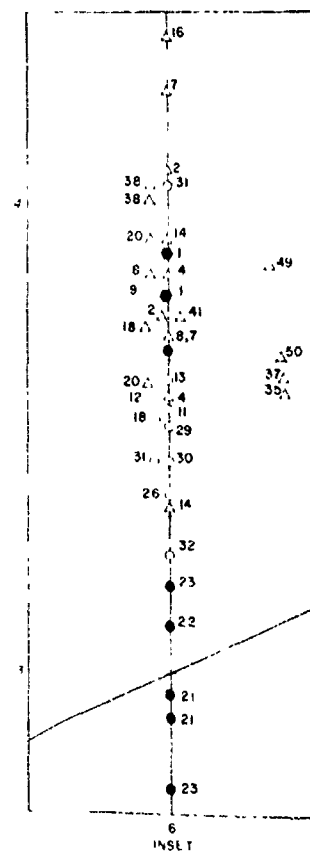
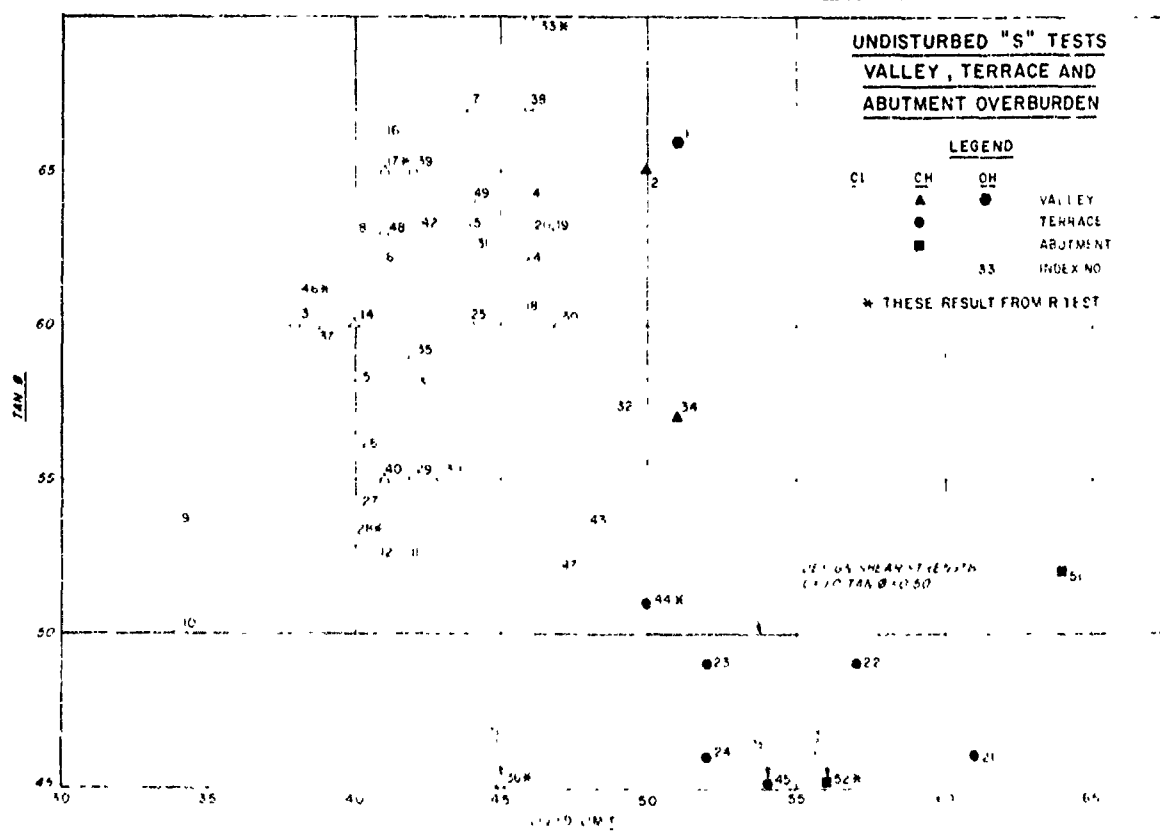
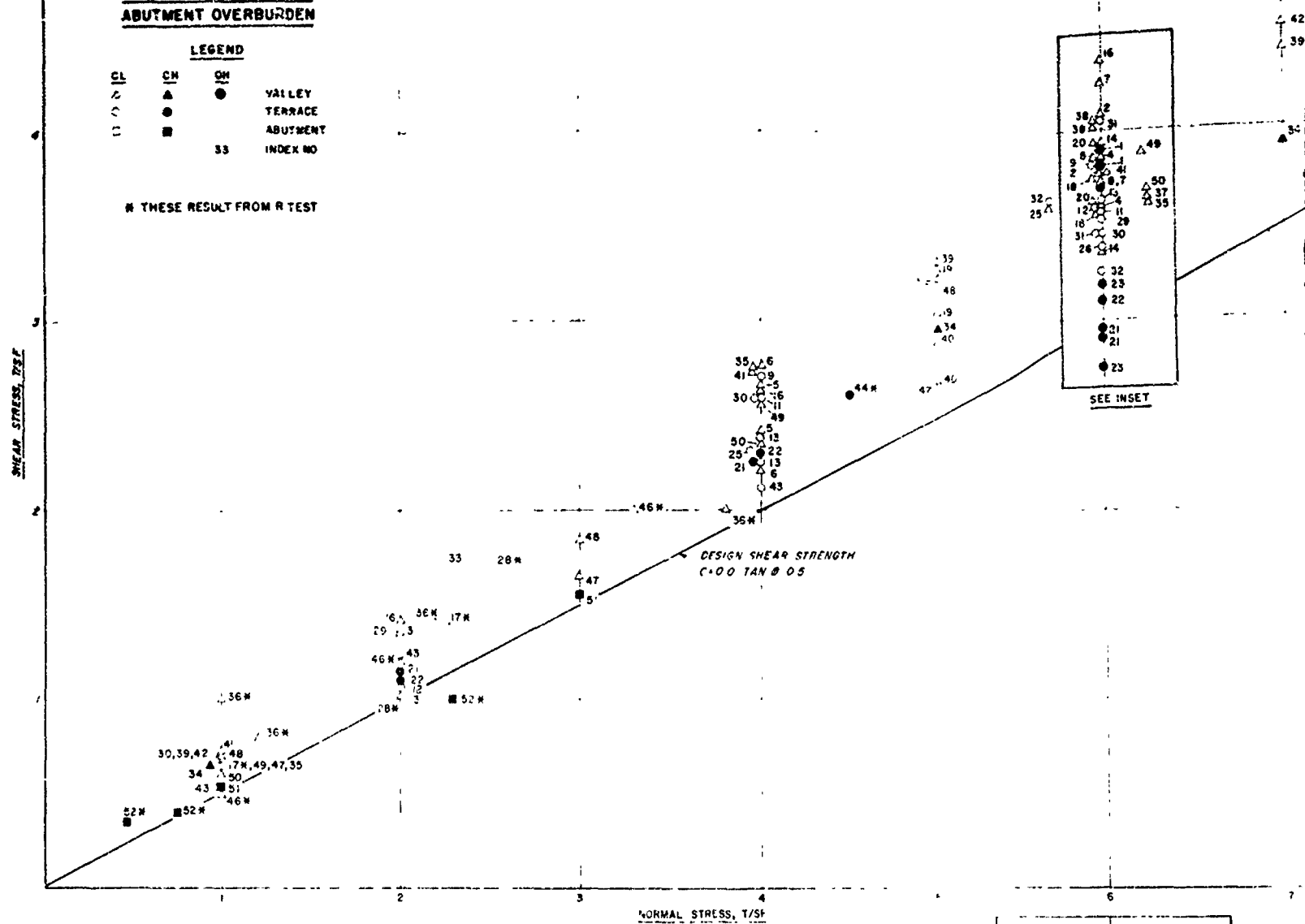
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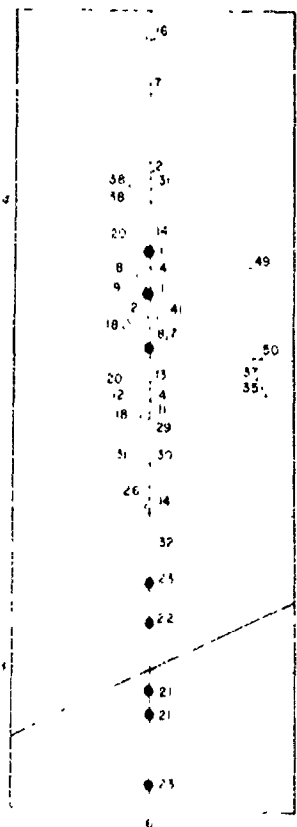
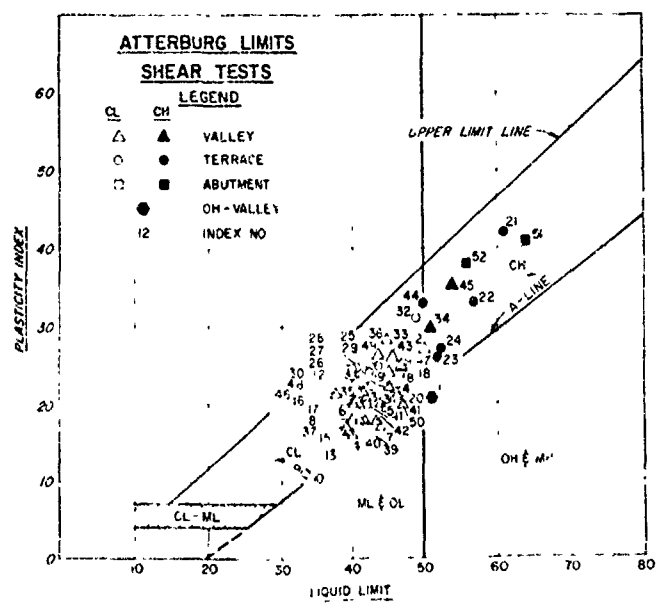
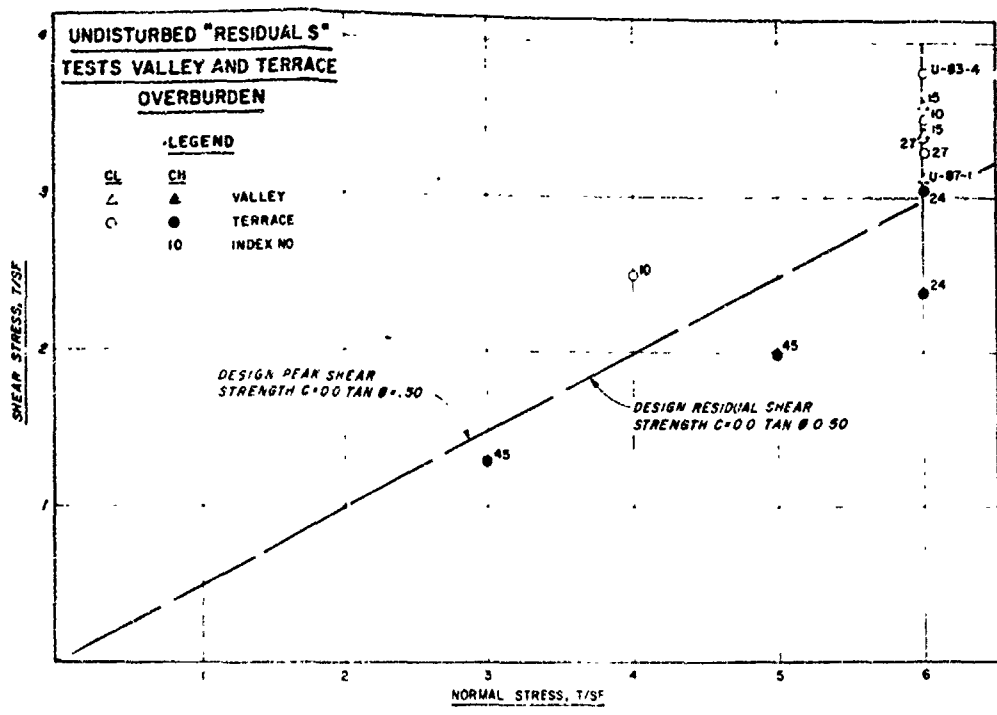
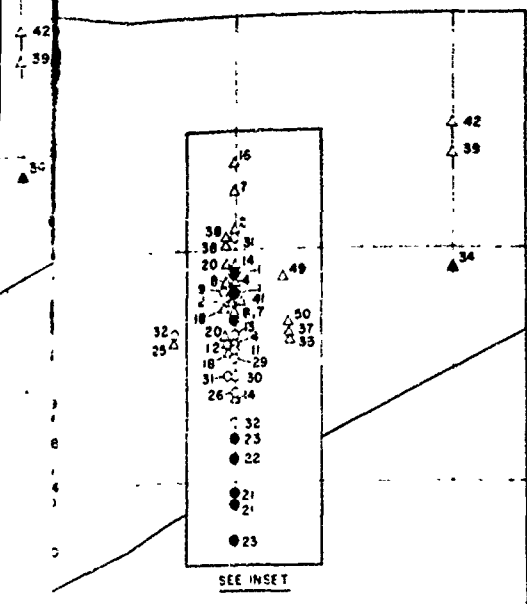
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ER: -3-1692

PLATE NO. 86G

4 THESE RESULT FROM R TEST





SYMBOL

DESCRIPTION

RIVER/ONS

DATE

APPROVED

LITTLE BLUE RIVER MISSOURI

## LONGVIEW LAKE

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

TEST DATA SUMMARY

UNDISTURBED FOUNDATION OVERBURDEN

S-TESTS, PLOTS

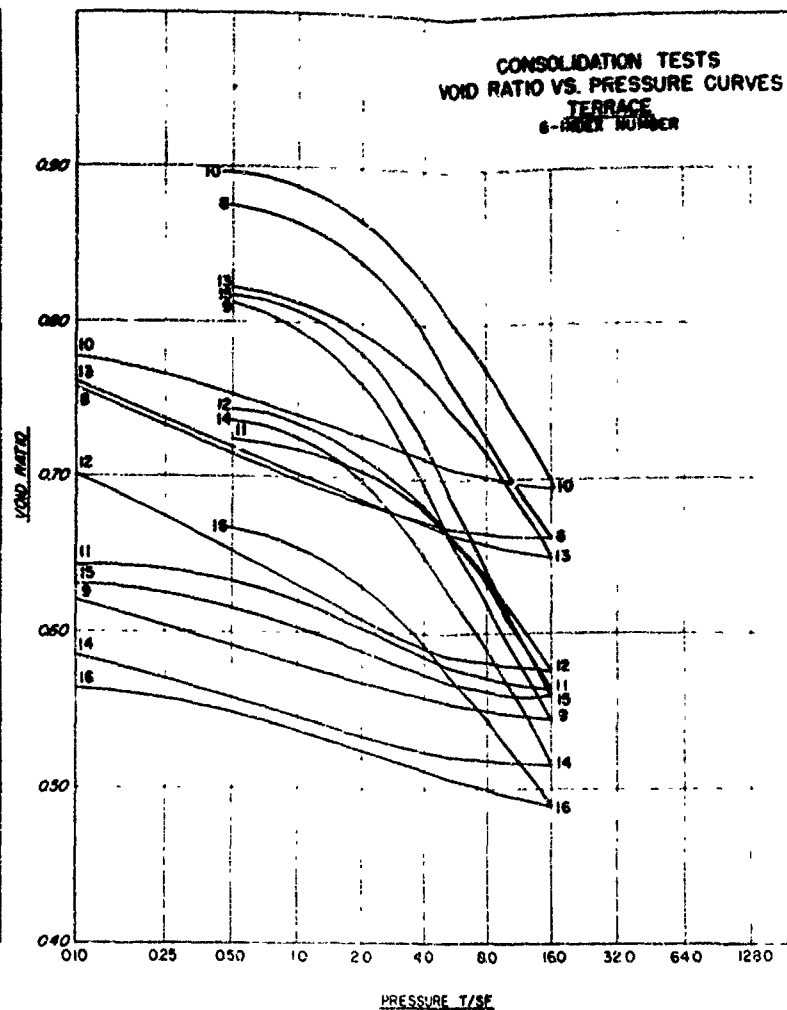
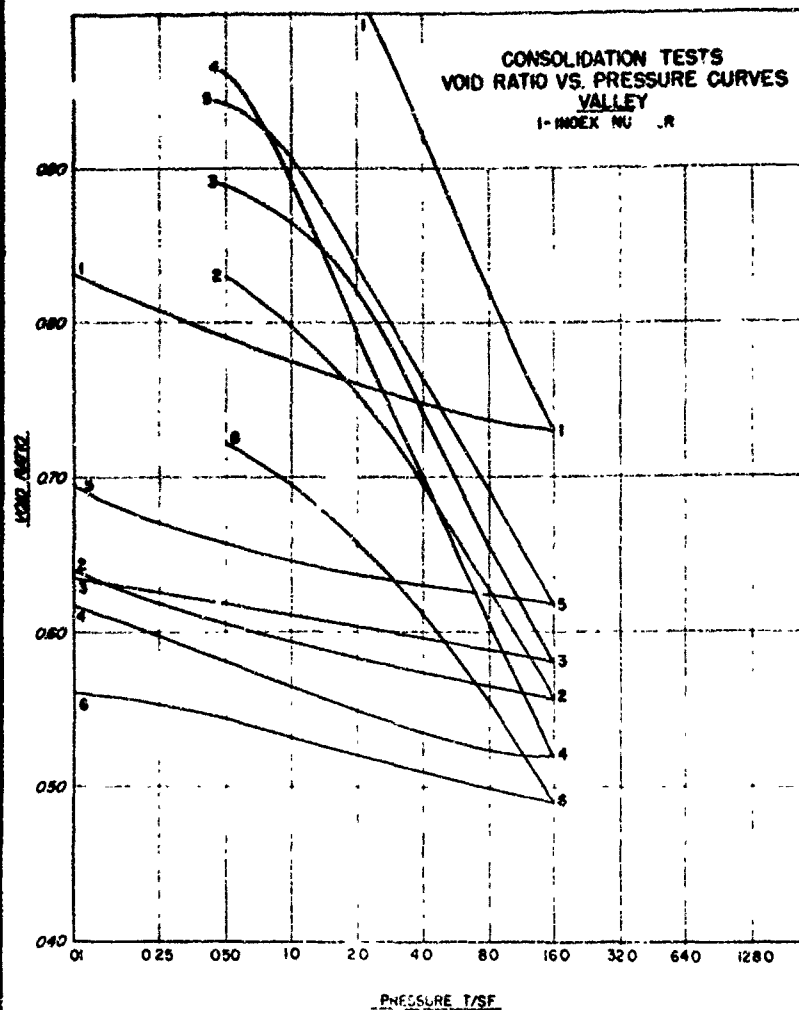
Sheet No. 1

Scale as shown

U.S. ARMY

JUNE 1978

RBL-3-1693



**CONSOLIDATION TEST DATA  
VALLEY (DAM AXIS STA 90+00 - 97+00)**

INDEX NUMBER	HOLE NUMBER	SAX NUMBER	DEPTH IN	TSF	TSF	TSF	TSF	TSF	TSF	TSF	TSF	TSF	TSF
1	U 10A	5	3.5	14	0.10	0.046	0.31	0.0245	0.0175	0.0272			
2	U 10A	4	4.5	14	0.10	0.046	0.31	0.0245	0.0175	0.0272			
3	U 10B	4	2.0	14	0.10	0.046	0.31	0.0245	0.0175	0.0272			
4	U 10A	5	3.5	14	0.10	0.046	0.31	0.0245	0.0175	0.0272			
5	U 10A	5	3.5	14	0.10	0.046	0.31	0.0245	0.0175	0.0272			
6	U 10A	5	3.5	14	0.10	0.046	0.31	0.0245	0.0175	0.0272			

**CONSOLIDATION TEST DATA  
TERRACE (DAM AXIS STA 97+00 - 100+00)**

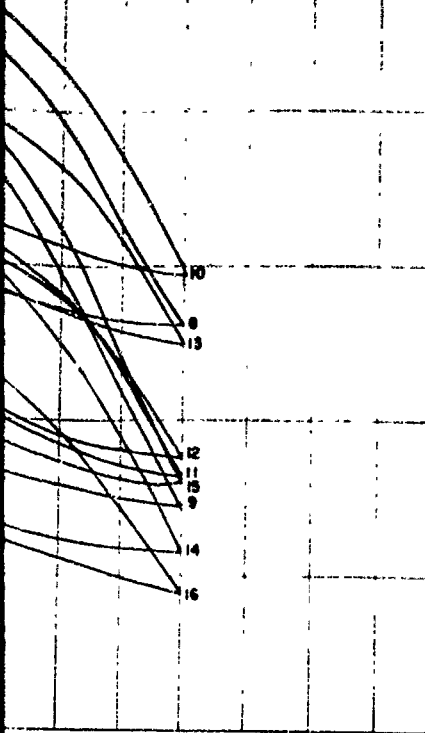
INDEX NUMBER	HOLE NUMBER	SAX NUMBER	SAMPLE DEPTH	TSF	TSF	TSF	TSF	TSF	TSF	TSF	TSF	TSF	TSF
8	U 10A	4	6.9 - 7.0	43	0.38	2.7							
9	U 10A	4	12.5 - 12.6	40	0.31	1.9							
10	U 10A	4	17.2 - 17.4	45	0.38	2.8							
11	U 10A	4	6.3 - 6.5	53	0.39	5.9							
12	U 10A	4	7.8 - 8.0	54	0.48	5.7							
13	U 10A	4	15.1 - 15.4	46	0.92	4.5							
14	U 10A	4	9.2 - 9.4	40	0.16	2.7							
15	U 10A	4	19.0 - 19.1	46	1.19	2.7							
16	U 10A	4	2.3 - 3.2	44	0.19	2.6							



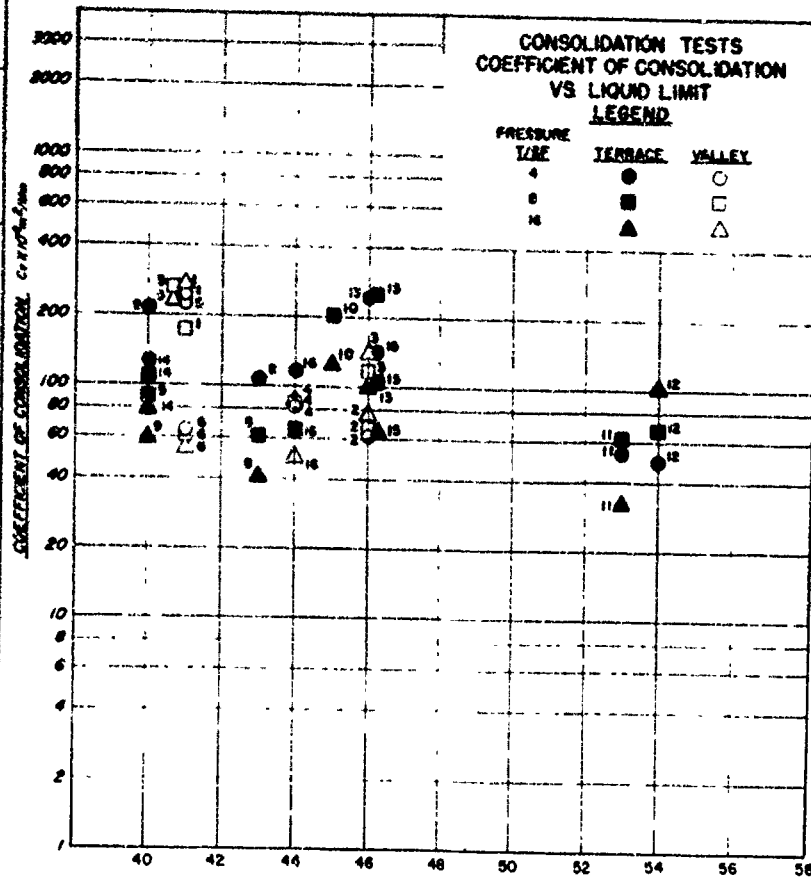
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# CONSOLIDATION TESTS VPO RATIO VS. PRESSURE CURVES TERRACE S-SHEET NUMBER

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PRESSURE 1/5F



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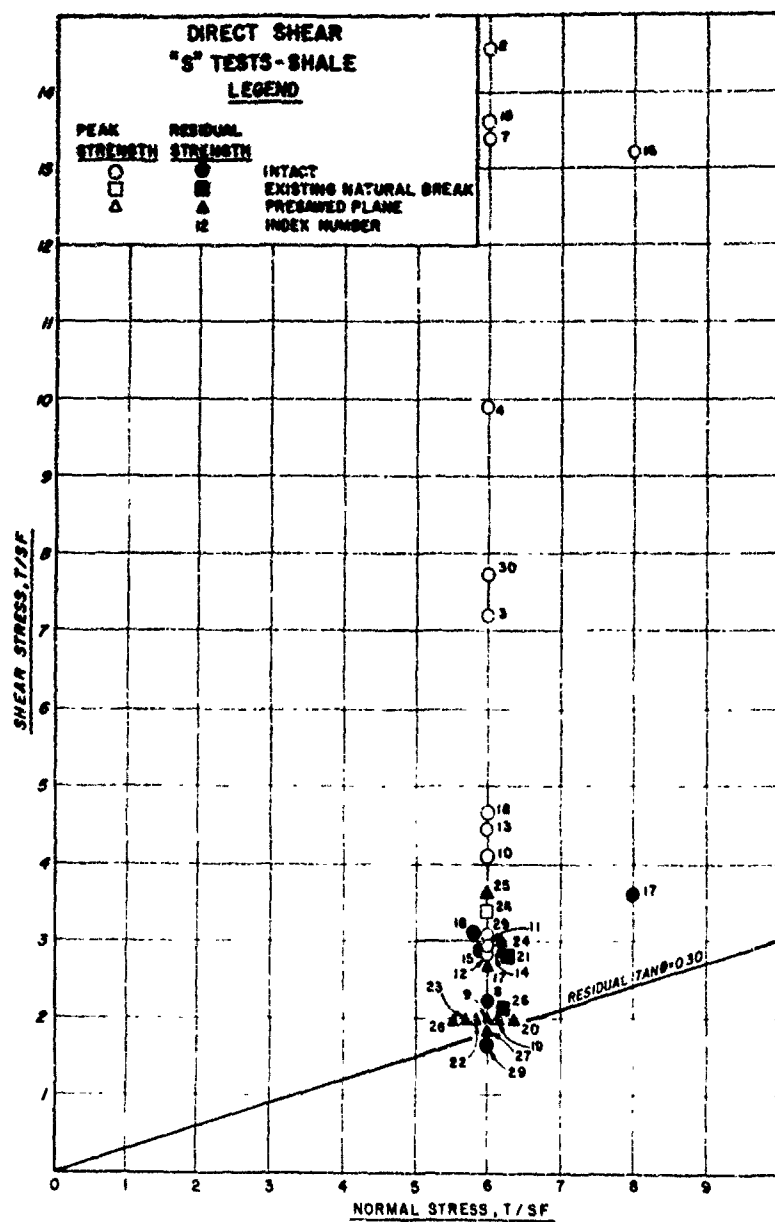
CONSOLIDATION TEST DATA  
TEST PLACE: DAM SITE #77 97+00 - 99+50

SHEET	WATER NUMBER	SAMPLE DEPTH	L	VPO		PL		e <sub>0</sub>		C <sub>c</sub>		C <sub>v</sub> (cm <sup>2</sup> /min)		
				1/5 F	1/5 F	1/5 F	1/5 F	1/5 F	1/5 F	1/5 F	1/5 F	1/5 F		
13	6	6.5 - 10	43	0.36	2.7	0.887	0.640	0.142	0.22	0.0107	0.0061	0.0042		
13	7	12.5 - 17.9	40	0.71	1.9	0.878	0.544	0.028	0.75	0.0213	0.0089	0.0059		
13	9	11 - 14	45	0.98	2.8	0.852	0.658	0.036	0.24	---	0.0204	0.0121		
14	8	4.3 - 6.5	53	0.39	5.9	0.745	0.565	0.034	0.22	0.0054	0.0061	0.0032		
14	5	2 - 4.6	54	0.48	5.7	0.744	0.576	0.060	0.21	0.0049	0.0066	0.0102		
14	2	5.1 - 5.4	46	0.92	4.5	0.845	0.640	0.051	0.24	0.0245	0.0245	0.0102		
14	5	5 - 7.4	42	0.56	2.2	0.749	0.513	0.033	0.25	0.0229	0.0110	0.0090		
14	1	5 - 15	46	1.19	2.2	0.841	0.540	0.032	0.21	0.0140	0.0102	0.0064		
14	2	5 - 4.4	44	0.19	2.6	0.687	0.493	0.032	0.17	0.0112	0.0061	0.0049		

SYM

DESCRIPTION  
REVISIONS  
LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
EMPALEMENT CRITERIA AND  
PERFORMANCE REPORT  
TEST DATA SUMMARY  
FOUNDATION OVERBOLT  
CONSOLIDATION TESTS

WWR



DIRECT SHEAR "S" TESTS											
TEST NUMBER	HOLE NUMBER	BOX NUMBER	SAMPLE DEPTH	INITIAL CONDITION			TEST RESULTS				
				DRY (LB/FT <sup>3</sup> )	WETNESS CONTENT	SATURATION	NORMAL STRESS	MAX. SHEAR STRESS (T/SF)	PEAK TAN $\phi$	RESIDUAL SHEAR STRESS (T/SF)	RESID. TAN $\phi$
1	DC-32	1	22.6-23.1	132		27	6		1.67		
2	DC-32	1	22.6-23.1	132		35	6		1.67		
3	DC-32	3	40.1-40.6	140	0	38	6	7.22	1.2		
4	DC-32	3	40.1-40.6	140	0	38	6	9.90	1.06		
5	DC-32	5	60.9-60.9	142		36	6		1.67		
6	DC-32	5	60.9-60.9	142		36	6		1.67		
7	AC-46	13	31.7-32.5	136.2	1.2	39	6	13.40	2.22		
8	AC-46	13	31.7-32.5	139.2	0.3	38	6	14.56	2.42	2.21	
9	AC-46	13	31.7-32.5	136.4	5.5	42	6			2.00	
10	C-56 <sup>0.0</sup>	1	24.0-24.0	116.3	15.0	38	6	4.16	0.0		
11	C-56 <sup>0.0</sup>	2	24.0-24.0	119.0	11.8	32	6	2.96	0.0		
12	C-56 <sup>0.0</sup>	11	50.2-50.2	120.6	9.6	36	6	2.85	0.0		
13	C-56	7	31.3-32.2	129.9	7.7	31	6	0.44	7.6		
14	C-41	2	20.6-20.5	130.0	5.2	44	6			2.00	
15	C-41	2	20.6-20.5	141.1	6.3	36	6	13.6	2.27	2.93	
16	C-41	2	20.6-20.5	142.0	5.8	36	6	13.2	1.95	3.62	
17	C-41	9	46.4-47.9	132.1	6.7	35.6	6			2.79	
18	C-41A	2	33.4-34.7	141.0	3.6	40.3	6	4.64	7.6	3.12	
19	UC-164	9	19.3-19.6	132.0	9.5	32	6			2.0	
20	UC-164	9	19.3-19.6	132.0	9.5	32	6			2.1	
21	UC-164	10	20.5	134	9.5	107	6			2.04	
22	UC-164	10	20.7-21.0	134.5	9.5	99	6			2.1	
23	UC-164	10	20.7-21.0	134.5	9.5	99	6			2.0	
24	UC-165	1	17.4-19.5	119.9	12.8	43	6			2.4	
25	UC-165	1	17.7-19.5	151.2	2.6	55	6			1.67	
26	UC-165	3	20.8-22.4	150.4	2.6	47	6	3.42	5.7	2.13	
27	DC-167	1	14.8	137.0	0	31	6			1.9	
28	DC-167	1	15.0	137.0	0	32	6			2.0	
29	UC-168	1	14.9	123	14	95	6	3.07	5.1	1.7	
30	UC-168	4	19.0-20.8	126.4	4.5	36	6	7.77	1.29		

TESTED SAMPLES (SEE LAST PAGE) WERE 10 TO 10.5 FT  
4 IN. DIAM. AREA

RESIDUAL TAN 0		DIRECT SHEAR TESTS						TYPE OF SHEAR SURFACE	GEOLOGIC MEMBER
STATION	TEST NO.	TEST RESULTS					RESIDUAL TAN 0		
		NORMAL STRESS (PSF)	MOISTURE CONTENT (%)	PEAK TAN 0	RESIDUAL SHEAR STRESS (PSF)	RESIDUAL TAN 0			
3	37	6		1.67				INTACT	PLEASANTON SHALE
	37	6		1.67				INTACT	PLEASANTON SHALE
	38	6	7.22	1.2				INTACT	PLEASANTON SHALE
	38	6	9.75	1.66				INTACT	PLEASANTON SHALE AND SANDSTONE
	39	6		1.67				INTACT	PLEASANTON SHALE
	39	6		1.67				INTACT	PLEASANTON SHALE
	79	6	13.40	2.23				INTACT	P. Pleasanton Shale
	78	6	14.56	2.42	2.25	31		INTACT	P. Pleasanton Shale
	63	6				2.00	33	PRESERVED	P. Pleasanton Shale
	90	6	4.36	89				INTACT	SEA SHALE
4	72	6	2.96	49				INTACT	SEA SHALE
	76	6	2.85	48				INTACT	STANN GALESDURG 40
	71	6	4.44	74				INTACT	PLEASANTON SHALE
	64	6				2.80	48	PRESERVED	PLEASANTON SHALE
7	84	6	13.6	2.27	7.93	49		INTACT	PLEASANTON SHALE
	81	6	13.2	1.65	3.53	45		INTACT	P. Pleasanton Shale
	65.6	6				2.79	47	PRESERVED	PLEASANTON SHALE
	48.0	6	4.67	75	3.11	52		INTACT	PLEASANTON SHALE
	92	6			2.0	33		PRESERVED	PLEASANTON SHALE
	92	6			2.1	35		PRESERVED	PLEASANTON SHALE
	100	6			2.00	46		EXISTING BREAK	PLEASANTON SHALE
	99	6			2.1	35		PRESERVED	PLEASANTON SHALE
	99	6			2.0	33		PRESERVED	PLEASANTON SHALE
	83	6			2.55	49		PRESERVED	PLEASANTON SILTSTONE
2	50	6			1.62	60		PRESERVED	PLEASANTON SILTSTONE
	45	6	3.42	51	2.13	34		EXISTING BREAK	PLEASANTON SILTSTONE
	93	6			1.6	30		PRESERVED	PLEASANTON SILTSTONE
	92	6			2.0	33		PRESERVED	PLEASANTON SILTSTONE
	99	6	3.0	51	1.1	28		INTACT	PLEASANTON SHALE
	36	6	2.11	1.29				INTACT	PLEASANTON SHALE

17 OF 10 13



DESCRIPTION  
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LITTLE BLUE RIVER MISSOURI  
**LONGVIEW LAKE**  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
TEST DATA SUMMARY  
FOUNDATION SHALES

In 1 sheet  
CORPS OF ENGINEERS  
KANSAS CITY DISTRICT

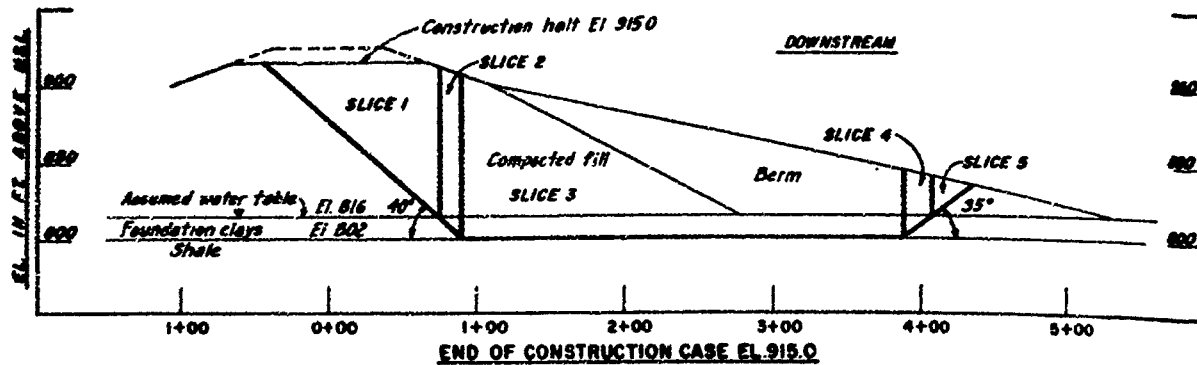
Sheet No 1

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U.S. ARMY  
JUNE 1976

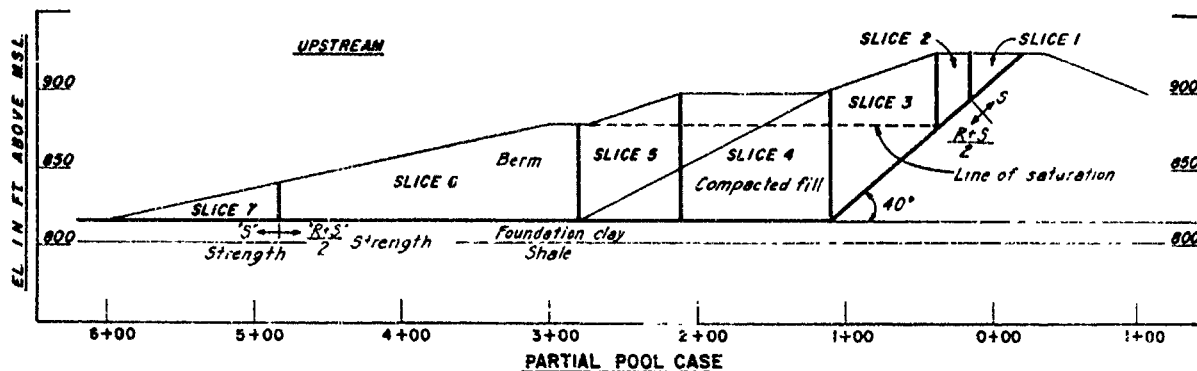
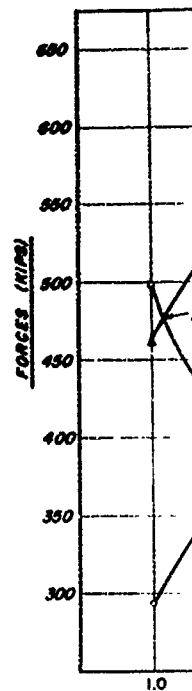
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RBL-3-1695

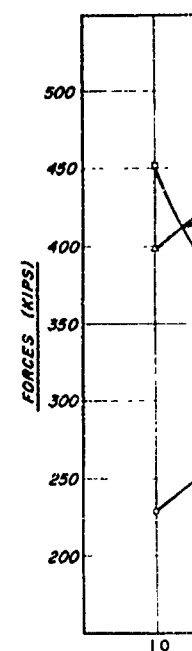
PLATE NO. 86J



END OF CONSTRUCTION CASE EL 915.0												
SLICE	C <sup>1</sup> STRENGTH		L (FT)	W <sub>EFF</sub> (KIPS)	W <sub>TOTAL</sub> (KIPS)	TAN $\phi$	H	F.S.	1.0	1.34	1.80	2.0
	CRUSH	TAN $\phi$										
1	180	0	118	682.1	682.1	.09	0	Driving	195.9	281.3	308.2	344.3
2	120	0	15	162.2	167.7	.09	0	Driving	98.1	105.4	107.6	112.4
3	120	0	300	2187.2	2408.1	0	0	Resisting	360.0	268.7	240.0	180.5
4	120	0	18	59.1	65.7	0	0	Resisting	73.6	65.4	62.8	57.5
5	90	0	28	38.5	38.5	0	0	Resisting	64.5	55.0	52.0	45.7



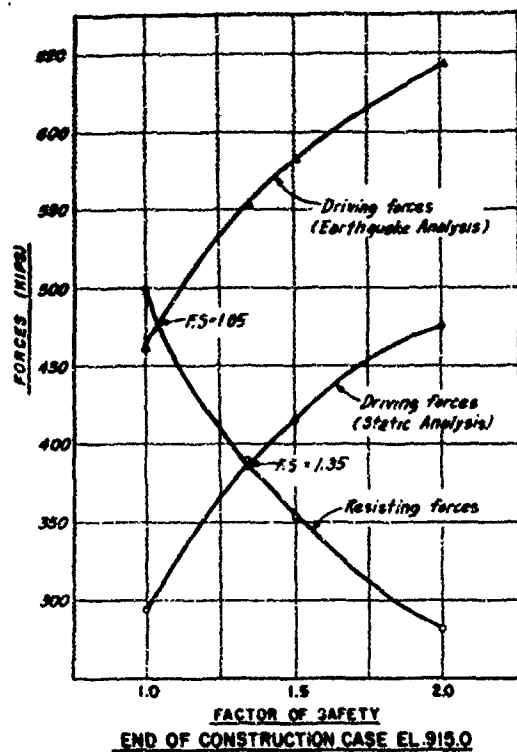
PARTIAL POOL CASE EL.878													
SLICE	S <sup>1</sup> STRENGTH		S <sup>2</sup> STRENGTH		L (FT)	W <sub>EFF</sub> (KIPS)	W <sub>TOTAL</sub> (KIPS)	TAN $\phi$ /2	$\frac{1}{2}$ H	F S EARTH FORCES (KIPS)	1.0	1.53	2.0
	C (KIPS)	TAN $\phi$	C (KIPS)	TAN $\phi$									
1	0	36			56	607	607	09	0	Driving	217	293	331
2			20	28	225	1016	1016	09	0	Driving	383	509	570
3			20	28	72	431.8	573.8	09	0	Driving	1690	2207	2456
4			20	28	102	620.0	1002.3	0	0	Resisting	1940	1268	970
5			20	28	68	797.7	529.4	0	0	Resisting	982	642	491
6			10	25	202	475.5	951.0	0	0	Resisting	1392	910	696
7	0	30			117	73.9	147.8	0	0	Resisting	222	145	711



PARTIAL POOL CASE EL 878			
FACTOR OF SAFETY	1.0	1.53	2.0
RESISTING FORCES (KIPS)	4535	2969	2264
DRIVING FORCES	2290	3009	3357
STATIC ANALYSIS (KIPS)	3977	4696	5044

SUMMARY - PARTIAL POOL CASE			
TYPE OF ANALYSIS	HAND ANALYSIS FS	COMPUTER FS	REQUIRED FS
STATIC	1.52	1.53	1.50
EARTHQUAKE	1.11		1.00

$\Sigma$  Driving forces (Earthquake Analysis) =  
 $\Sigma$  Driving forces (Static analysis)  
 $\Delta$  Driving forces  
 Where  
 $\Delta$  Driving forces =  $\Psi \cdot W_T$   
 $= 0.05 \cdot 3374.5 \text{ K}$   
 $= 168.7 \text{ K}$



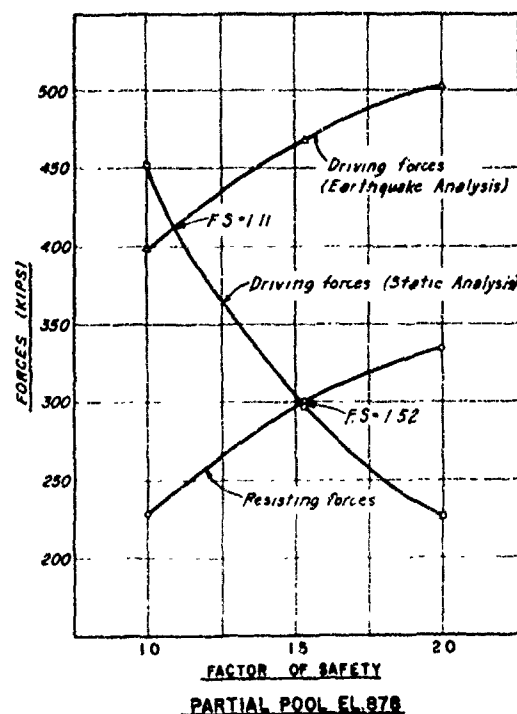
SUMMARY - END OF CONSTRUCTION CASE			
TYPE OF ANALYSIS	HAND ANALYSIS F.S.	COMPUTER F.S.	REQUIRED F.S.
STATIC	1.35	1.34	1.30
EARTHQUAKE	1.05	—	1.00

END OF CONSTRUCTION CASE EL 915.0				
FACTOR OF SAFETY	1.0	1.34	1.5	2.0
Σ RESISTING FORCES (KIPS)	498.1	389.1	354.8	283.2
Σ DRIVING FORCES (KIPS)	294.0	386.0	415.8	476.7
Σ EARTHQUAKE ANALYSIS (KIPS)	462.1	554.1	583.9	644.8

Σ Driving forces (Earthquake Analysis) =  
Σ Driving forces (static analysis) + Δ Driving forces

Where Δ Driving forces =  $\frac{P}{\Sigma W} \times W$   
=  $0.05 \times 3362.1$   
= 168.1 (Kips)

PHYSICAL SOIL CONSTANTS								
MATERIAL	UNIT WEIGHT		DESIGN SHEAR STRENGTHS					
	SAT.	DRAINED	C (KIPS)	TAN Ø	C (KIPS)	TAN Ø	C (KIPS)	TAN Ø
BERM	115	110	90	0.0	20	20	0.0	.30
COMPACTED FILL (HILLSIDE SLOPE)	125	115	180	0.0	40	.20	0.0	.36
FOUNDATION CLAY	115	110	120	0.0	40	.20	0.0	.50
SHALE - PEAK	140	140	—	—	—	—	0.0	.06



SYM

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LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

TYPICAL STABILITY ANALYSIS  
END OF CONSTRUCTION AND PARTIAL POOL CASE

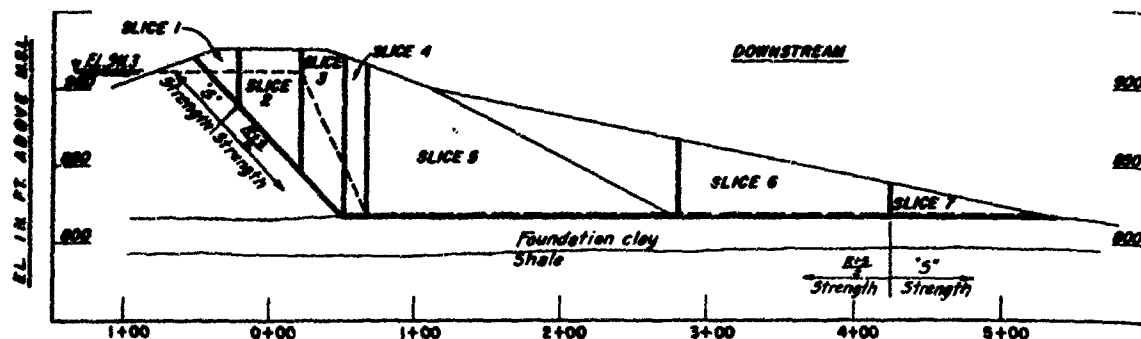
In 1 sheet  
CORPS OF ENGINEERS  
KANSAS CITY DISTRICT

Sheet No 1

Submitted  
Checked by  
Reviewed by  
RGD

Revised  
Checked by  
Reviewed by  
RGD

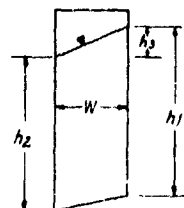
ED F.S.



STEADY SEEPAGE SPILLWAY CREST EL. 911.3

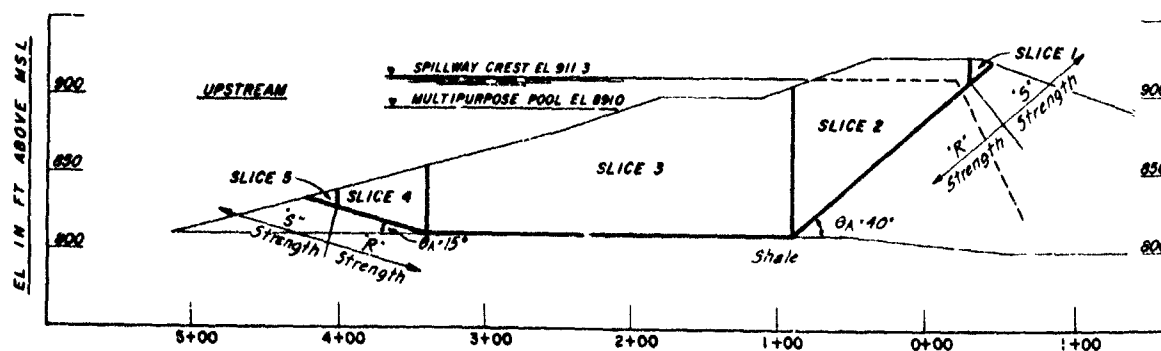
SLICE	"S" STRENGTH		"R" STRENGTH		L (FT)	W <sub>EFF</sub> (KIPS)	W <sub>TOTAL</sub> (KIPS)	TAN $\phi$	$\gamma_w$	EARTH FORCES (KIPS)	F.S.	1.0	1.5	2.0
	C (KIPS)	TAN $\phi$	C (KIPS)	TAN $\phi$										
1	0	.36			300	54.6	694	.09	0	Driving		24.7	31.7	35.7
2			.20	.28	42.5	177.9	206.4	.09	0	Driving		82.6	105.9	118.7
3			.20	.28	30.0	236.8	329.4	.09	185.3	Driving		29.2	32.0	335.9
4			.20	.28	17	182.2	201.6	0	38.1	Resisting		16.3	-1.8	-10.9
5			.20	.28	212	1728.1	1748.1	0	0	Resisting		526.3	350.8	263.1
6			.10	.25	72	565.8	577.2	0	0	Resisting		155.9	103.9	77.9
7	0	.30			107	131.7	139.8	0	0	Resisting		39.5	26.3	19.8
8														

Sample calculation for  $\gamma_w$  (Seepage Force)



$$\gamma_w = \frac{W}{A} \cdot i$$

Where  $\gamma_w$  = unit wt of water  
 $A = (w)(h_1 + h_2)$   
 $i$  = hydraulic gradient  
 $i = h_2/w$



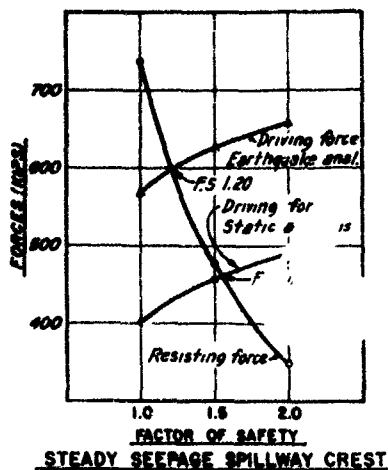
RAPID DRAWDOWN FROM SPILLWAY CREST EL. 911.3

SLICE	"S" STRENGTH		"R" STRENGTH		L (FT)	W <sub>EFF</sub> (KIPS)	TAN $\phi$	TAN $\phi$	$\gamma_w$	EARTH FORCES (KIPS)	F.S.	1.0	1.21	1.50	2.0
	C (KIPS)	TAN $\phi$	C (KIPS)	TAN $\phi$											
1	0	.50			15	4.3	.81	.09	0	Driving		3.4	4.4	5.5	6.7
2			.40	.20	11.9	458.9	.81	.09	29.3	Driving		231.3	262.3	292.0	324.4
3			.40	.20	250	1164.8	0	0	88.4	Resisting		244.6	186.8	133.6	78.1
4			.40	.20	62	108.7	.27	0	0	Resisting		82.1	72.5	63.9	55.0
5	0	.50			22	8.1	.27	0	0	Resisting		7.2	6.2	5.4	4.5

SUMMARY - I  
TYPE OF AN  
STATI

RAPID D  
FACTOR  
RESISTIN  
DRIVING

RAPID I



SUMMARY-STEADY SEEPAGE SPILLWAY CREST			
TYPE OF ANALYSIS	HAND ANAL. F.S.	COMP. F.S.	REQ'D F.S.
STATIC	1.53	1.53	1.50
EARTHQUAKE	1.20	—	1.0

STEADY SEEPAGE AT SPILLWAY CREST			
FACTOR OF SAFETY	1.0	1.5	2.0
Σ RESISTING FORCES (KIPS)	7380	4732	3499
Σ DRIVING FORCES (KIPS)	101.5	158.6	140.3
Σ EARTHQUAKE ANALYSIS (KIPS)	5691	6262	6579

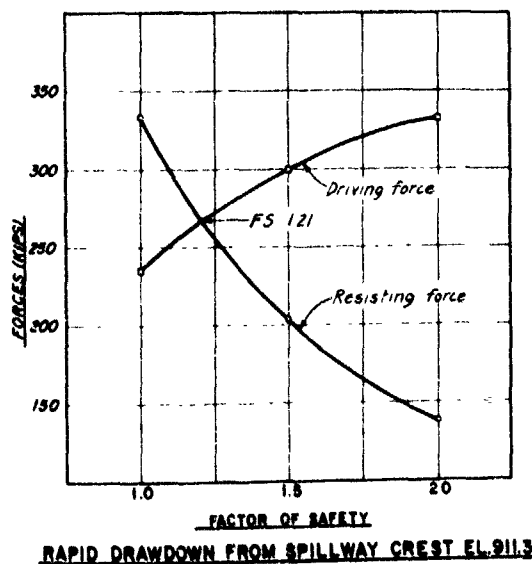
Driving forces (Earthquake Analysis) =  
Σ Driving forces (Static analysis) + Δ Driving forces

Where Δ Driving forces =  $\frac{1}{2}$  Total weight  
=  $0.5 \times 3351.9 K$   
= 1676 (Kips)

SUMMARY-RAPID DRAWDOWN FROM SPILLWAY CREST			
TYPE OF ANALYSIS	HAND ANAL. F.S.	COMPUTER F.S.	REQUIRED F.S.
STATIC	1.21	1.21	1.20

PHYSICAL SOIL CONSTANTS								
MATERIAL	UNIT WEIGHT PCF	DESIGN SHEAR STRENGTHS						
		Q	R		S			
	SAT	DRAINED	C COUPTS TAN δ	C COUPTS TAN δ	C COUPTS TAN δ	C COUPTS TAN δ	C COUPTS TAN δ	
BERM	1150	1100	90	0	20	20	0	50
COMPACTED FILL	1250	1150	180	0	40	20	0	36
FDN. CLAY	1150	1100	120	0	40	20	0	50
SHALE	1400	1400	—	—	—	—	0	0.6

RAPID DRAWDOWN FROM SPILLWAY CREST EL.911.3				
FACTOR OF SAFETY	1.0	1.21	1.50	2.0
Σ RESISTING FORCES	-333.9	-265.5	-202.9	-137.6
Σ DRIVING FORCES	2347	2667	297.5	331.1



#### NOTES:

1.  $\tan(\phi/2)$  for active wedge is assumed to be one half the average slope of the outer embankment.
2.  $\tan(\phi/2)$  for central block and passive wedge is assumed to be zero.
3. Seepage force equation is from book entitled Soil Mechanics, T.H. Wu, Section 3.12.
4.  $\gamma$  = seismic coefficient.

SYM

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LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
EMBANKMENT CRITERIA AND PERFORMANCE REPORT

TYPICAL STABILITY ANALYSIS  
RAPID DRAWDOWN AND STEADY SEEPAGE

In 1 sheet  
CORPS OF ENGINEERS  
KANSAS CITY DISTRICT

Sheet No. 1

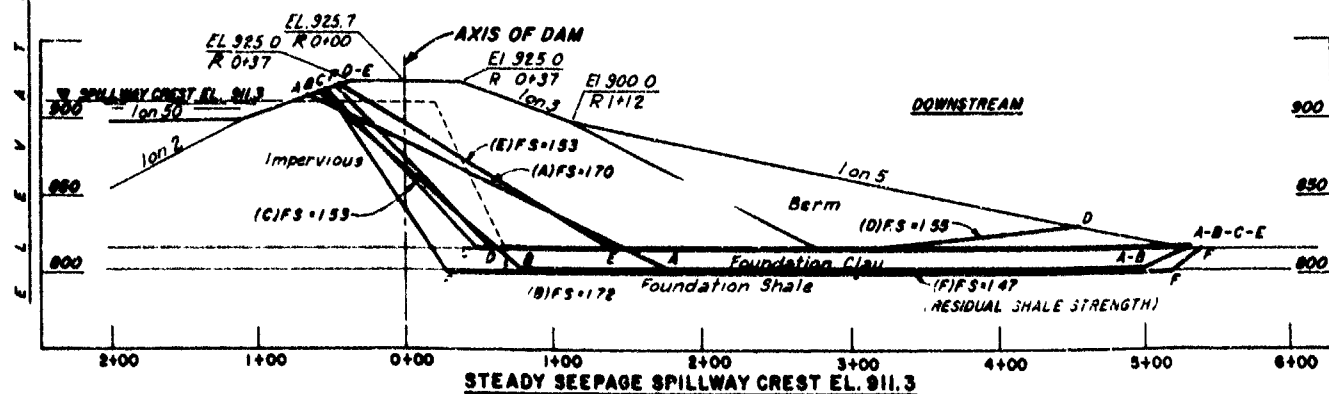
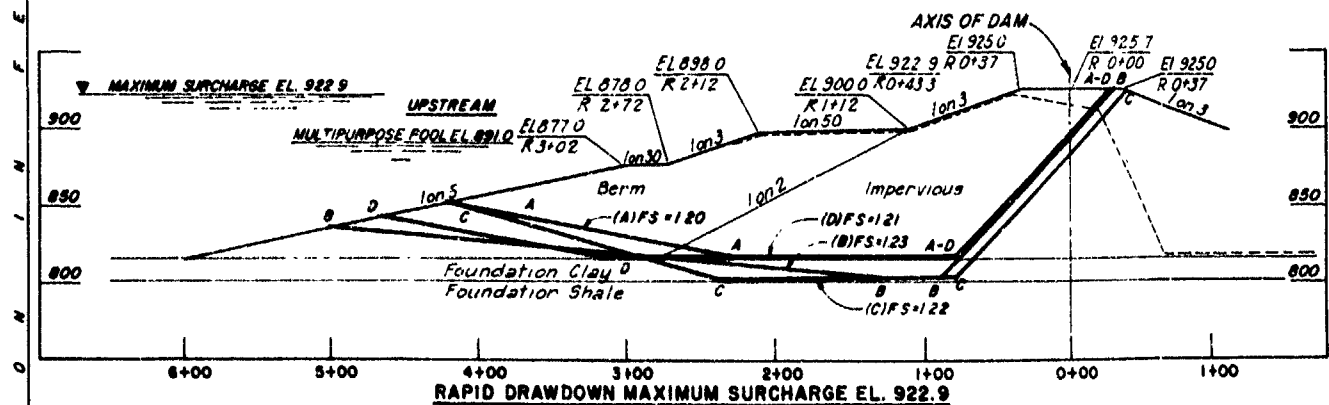
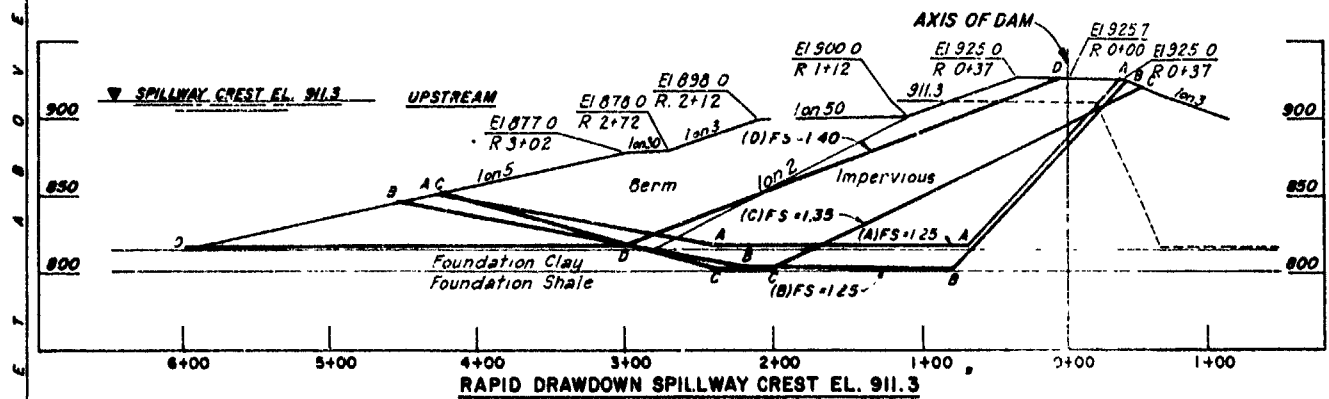
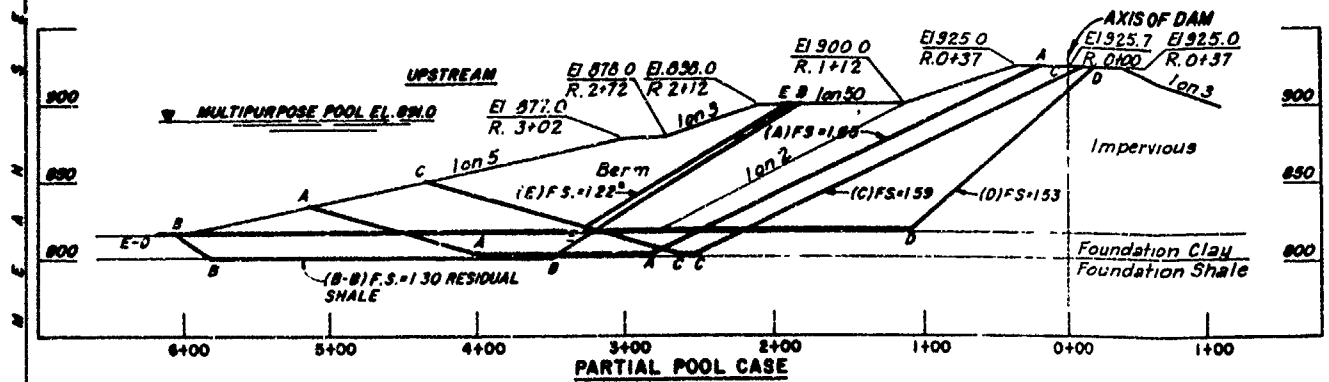
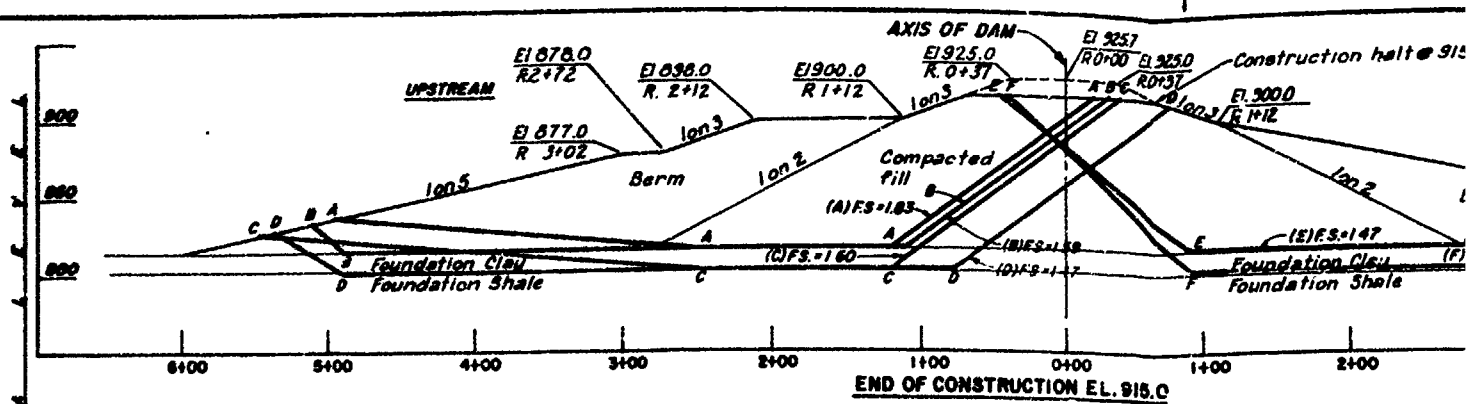
Scale as shown  
U. S. ARMY  
JUNE 1978

Submitted  
Checked by  
W.W.B.

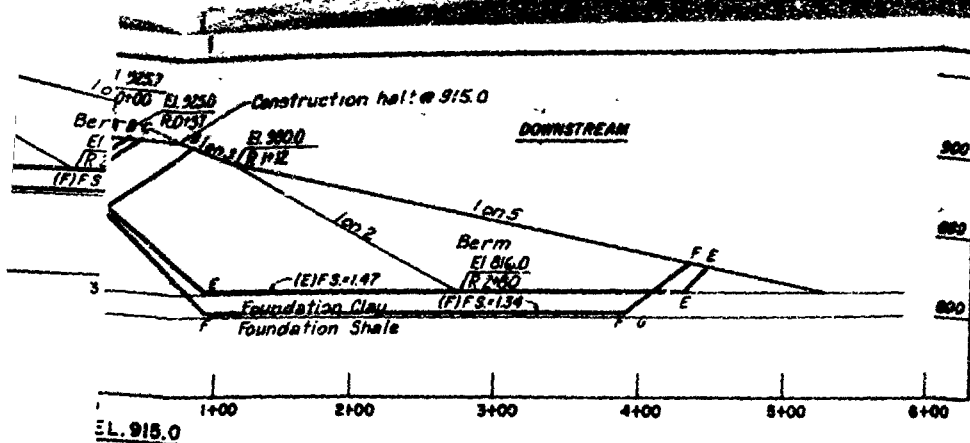
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Approved  
Checked by  
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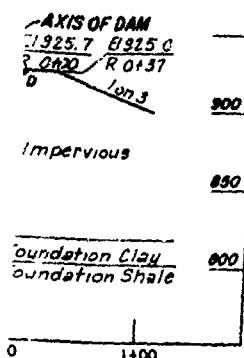
PLATE NO. 8H







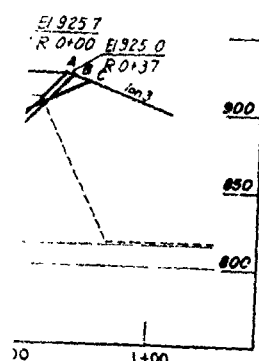
END OF CONSTRUCTION CASE			
SURFACE	FAILURE PLANE EL.	ELEVATION	FS
A	816	U/S	1.03
B	802	U/S	1.47
C	816	D/S	1.47
F	802	D/S	1.34



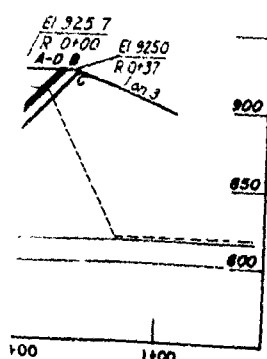
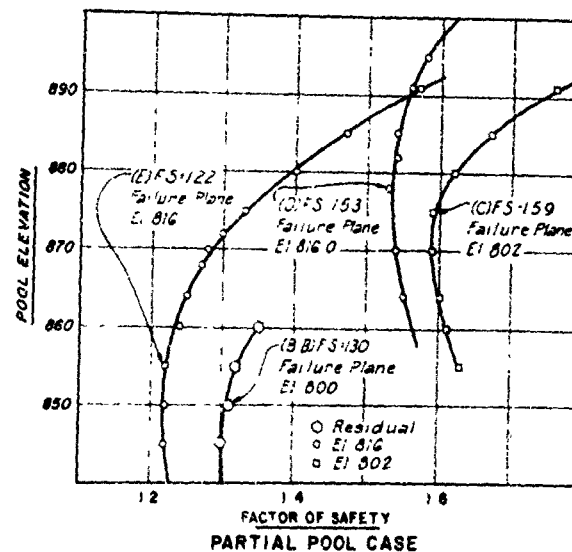
PHYSICAL SOIL CONSTANTS								
MATERIAL	UNIT WEIGHT PCF	DESIGN SHEAR STRENGTH						
		SAT	DRAINED	CU (PSI)	TAN $\phi$	CU (PSI)	TAN $\phi$	CU (PSI)
BERM	115	110	0.90	0.0	0.20	0.20	0.0	0.30
COMPACTED FILL (HILLSIDE BORROW)	125	115	1.80	0.0	0.40	0.20	0.0	0.36
COMPACTED FILL (VALLEY BORROW)	125	115	1.80	0.0	0.40	0.20	0.0	0.50
FOUNDATION CLAY	115	110	1.20	0.0	0.40	0.20	0.0	0.50
SHALE (PEAK)	140	140	—	—	—	—	0.0	0.60
SHALE (RESIDUAL)	140	140	—	—	—	—	—	0.30

NOTE: ALL SAFETY FACTORS COMPUTED UTILIZING HILLSIDE BORROW MATERIAL

STABILITY STUDIES USING RESIDUAL STRENGTHS FOR FOUNDATION SHALES		
SURFACE	CASE	FS
B-B	Partial pool	1.30
F-F	Steady seepage	1.47



RAPID DRAWDOWN FROM SPILLWAY CREST PEAK STRENGTHS			
SURFACE	FAILURE PLANE EL.	POOL LEVELS	FS
A	816	911.3/891	1.25
B	802	911.3/891	1.25
C	802	911.3/891	1.35
D	816	911.3/891	1.40



RAPID DRAWDOWN FROM MAXIMUM SURCHARGE PEAK STRENGTHS			
SURFACE	FAILURE PLANE EL.	POOL LEVELS	FS
A	816	922.9/891	1.20
B	802	922.9/891	1.23
C	802	922.9/891	1.22
D	816	922.9/891	1.21

STEADY SEEPAGE SPILLWAY CREST EL. 911.3 PEAK STRENGTHS			
SURFACE	FAILURE PLANE EL.	POOL LEVELS	FS
A	802	911.3/891	1.70
B	802	911.3/891	1.72
C	816	911.3/891	1.53
D	816	911.3/891	1.55
E	816	911.3/891	1.53

SYN DESCRIPTION REVISIONS DATE APP'D

LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
EMBANKMENT CRITERIA AND PERFORMANCE REPORT  
EMBANKMENT STABILITY ANALYSIS SUMMARY  
VALLEY SECTION

In 1 sheet  
CORPS OF ENGINEERS  
KANSAS CITY DISTRICT

Sheet No. 1

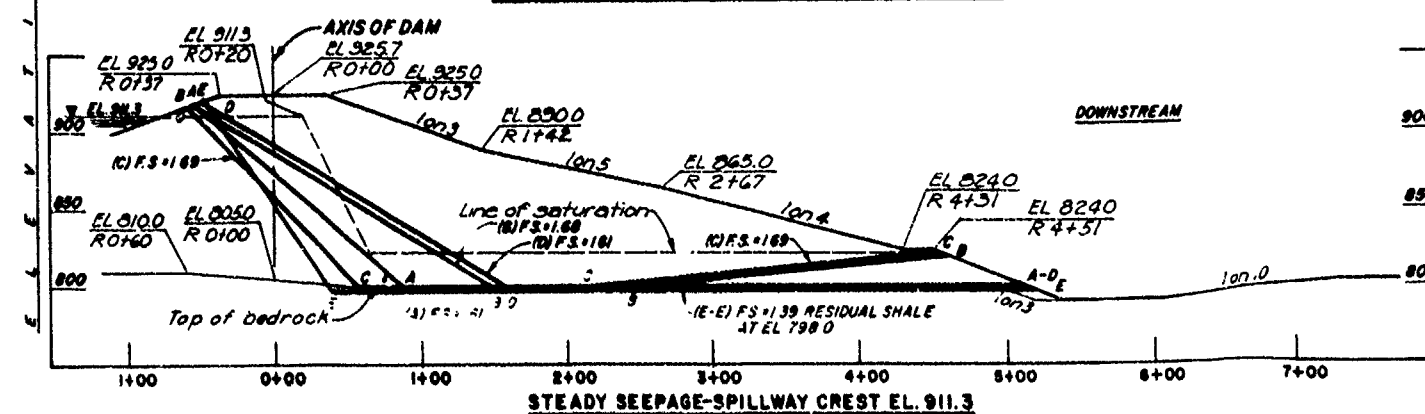
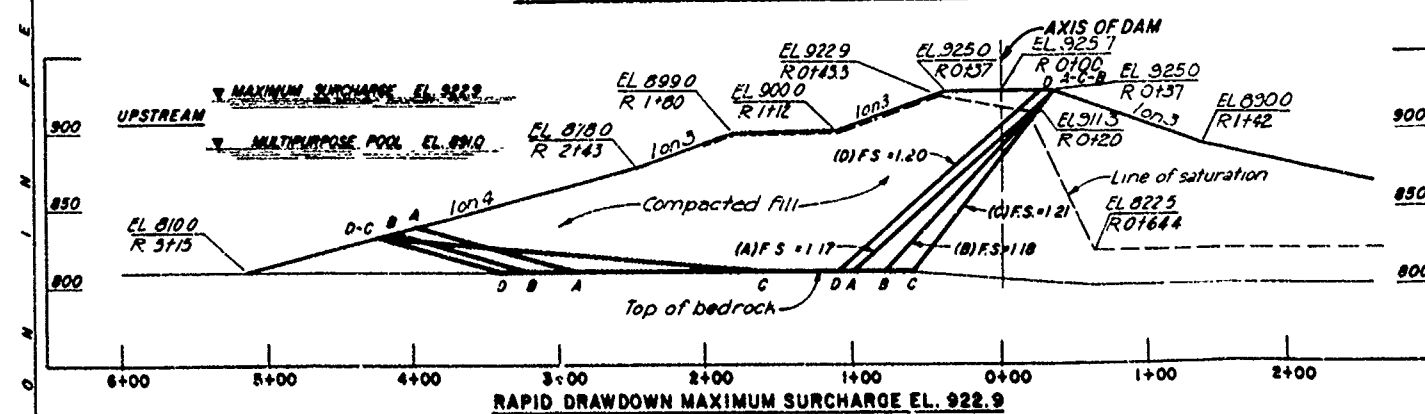
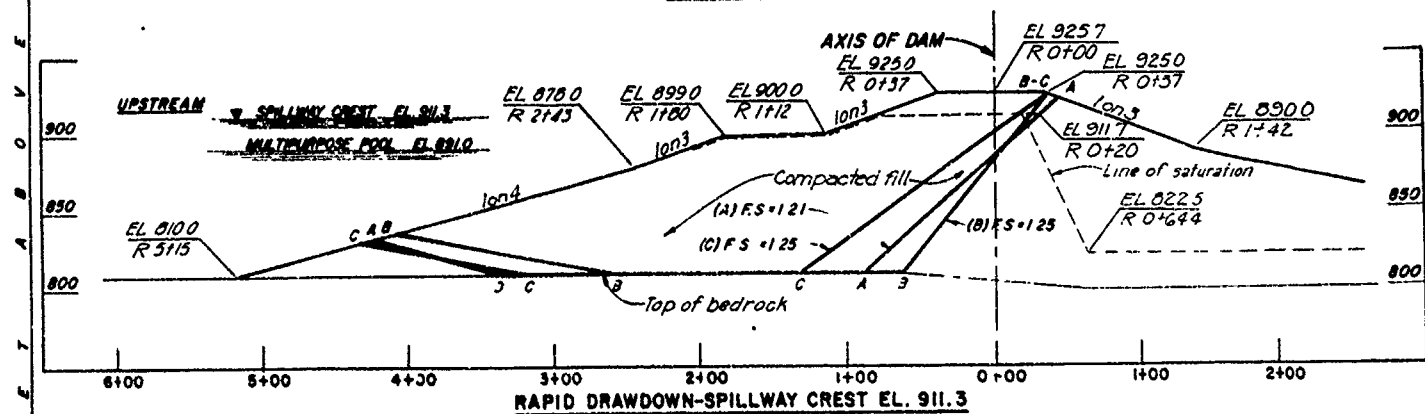
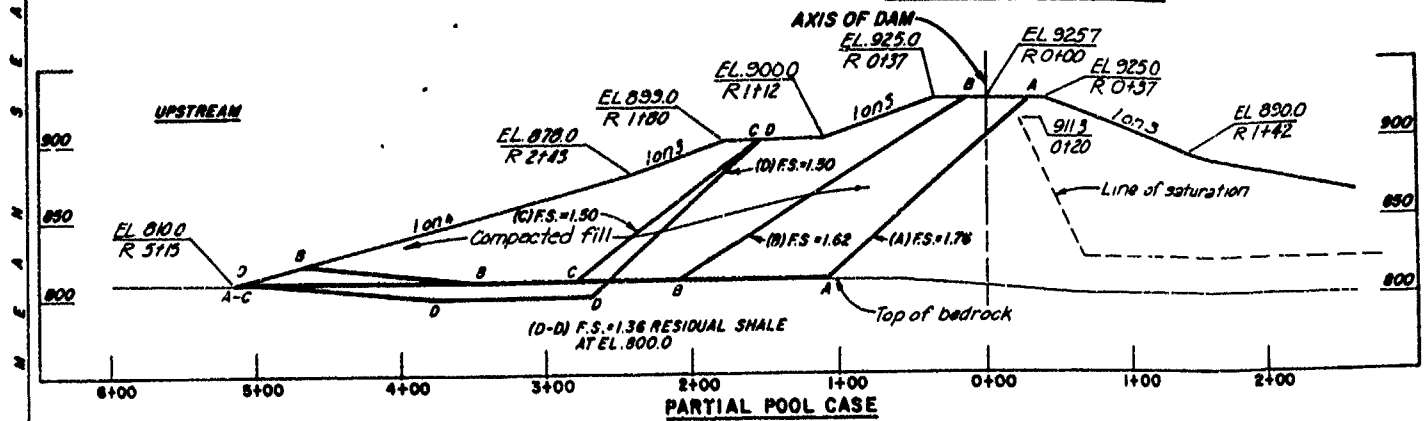
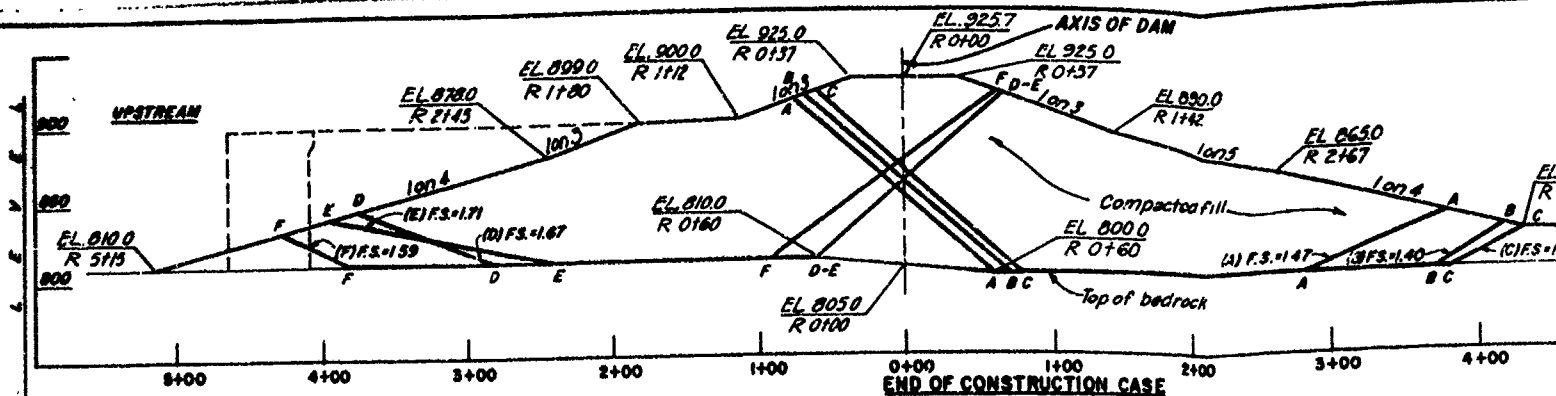
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U & ARMY  
JUNE 1976

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Checked by: [Signature]  
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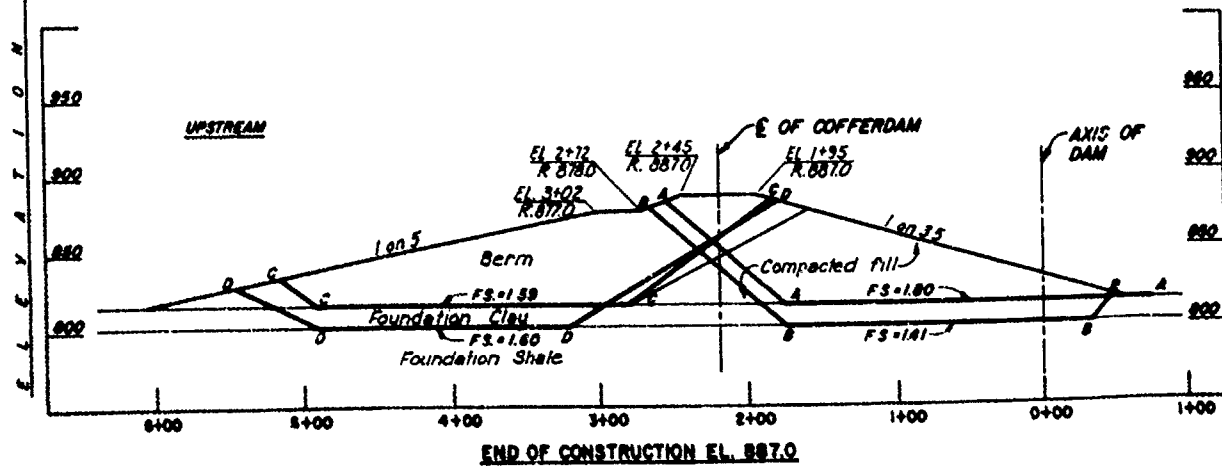
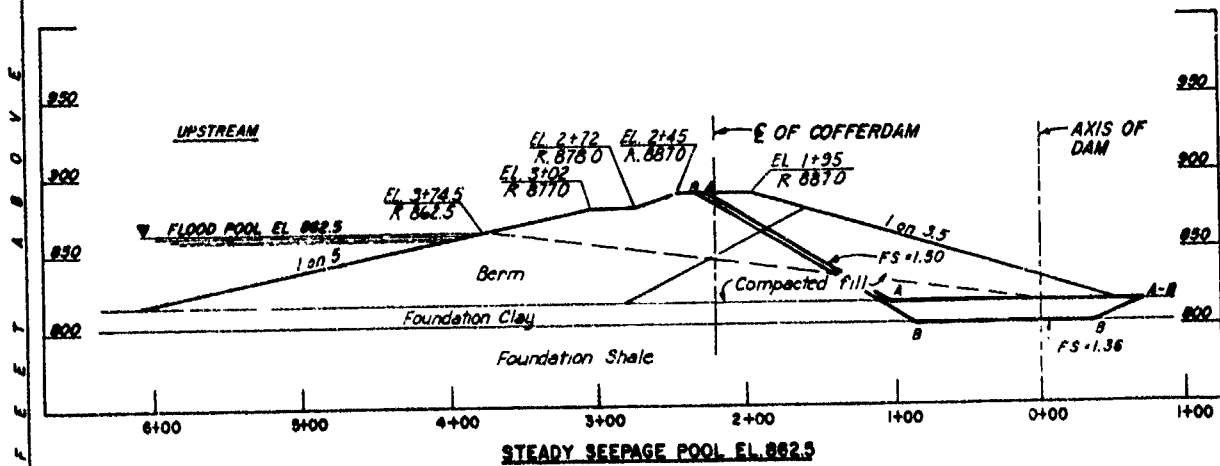
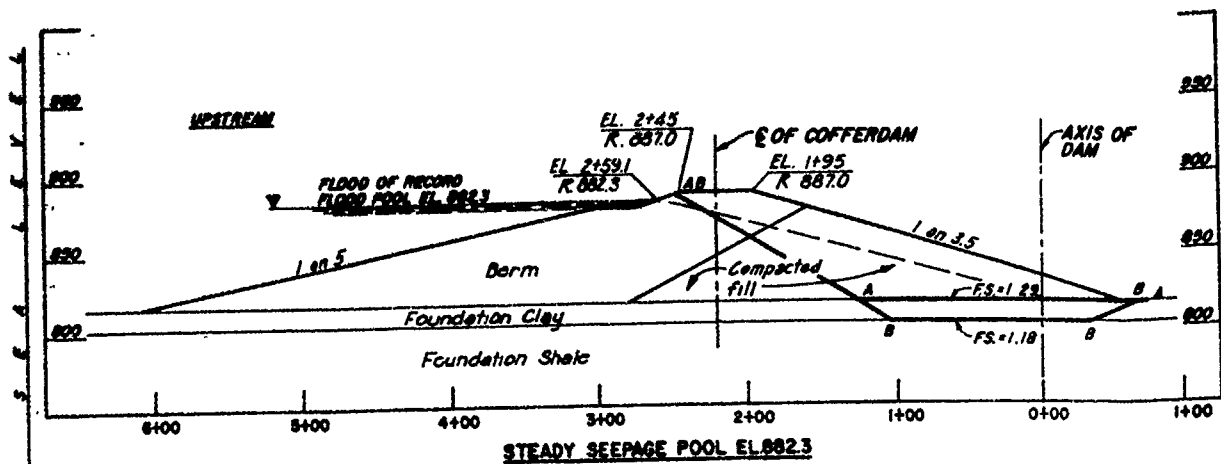
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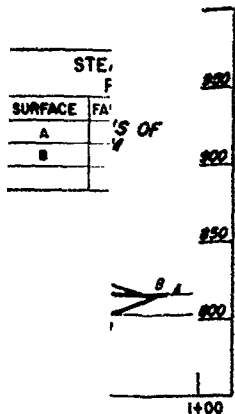
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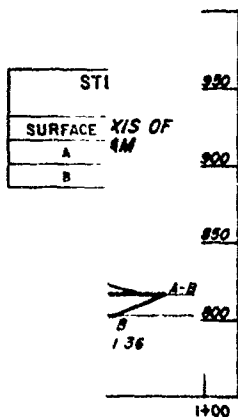






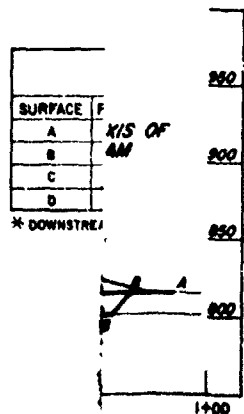
STEADY SEEPAGE CASE POOL EL. 882.3		
SURFACE	FAILURE PLANE EL.	F.S.
A	816	1.29
B	802	1.18

MATERIAL	UNIT WEIGHT PCF		DESIGN SHEAR STRENGTH							
			"Q"		"R"		"S"			
	SAT.	DRAINED	CD/PSI	TAN $\phi$	CD/PSI	TAN $\phi$	CD/PSI	TAN $\phi$	CD/PSI	TAN $\phi$
BERM	115	110	0.90	0	0.20	0.20	0.0	0.30		
COMPACTED FILL	125	115	1.80	0	0.40	0.20	.3	0.36		
FOUNDATION CLAY	115	110	1.20	0	0.40	0.20	0	0.50		
SHALE	140	140	—	—	—	—	0.0	0.60		



STEADY SEEPAGE CASE POOL EL. 882.5		
SURFACE	FAILURE PLANE EL.	F.S.
A	816	1.50
B	802	1.56

Notes  
 1. Elevation 882.3 is the maximum lake elevation for the all season flood of record routed through the outlet works  
 2. Elevation 882.5 is maximum elevation for the third flood of record for the period 1 Aug - 23 Feb routed through outlet works



END OF CONSTRUCTION POOL EL. 887.0			
SURFACE	FAILURE PLANE EL.	LOCATION	F.S.
A	816	D/S *	1.80
B	802	D/S *	1.41
C	816	U/S	1.59
D	802	U/S	1.60

\* DOWNSTREAM & OF COFFERDAM

SYN

DESCRIPTION  
REVISIONS

DATE APPD.

LITTLE BLUE RIVER, MISSOURI  
**LONGVIEW LAKE**  
 EMBANKMENT CRITERIA AND PERFORMANCE REPORT  
 EMBANKMENT STABILITY ANALYSIS  
 SUMMARY COFFERDAM

In 1 sheet  
 CORPS OF ENGINEERS  
 KANSAS CITY DISTRICT

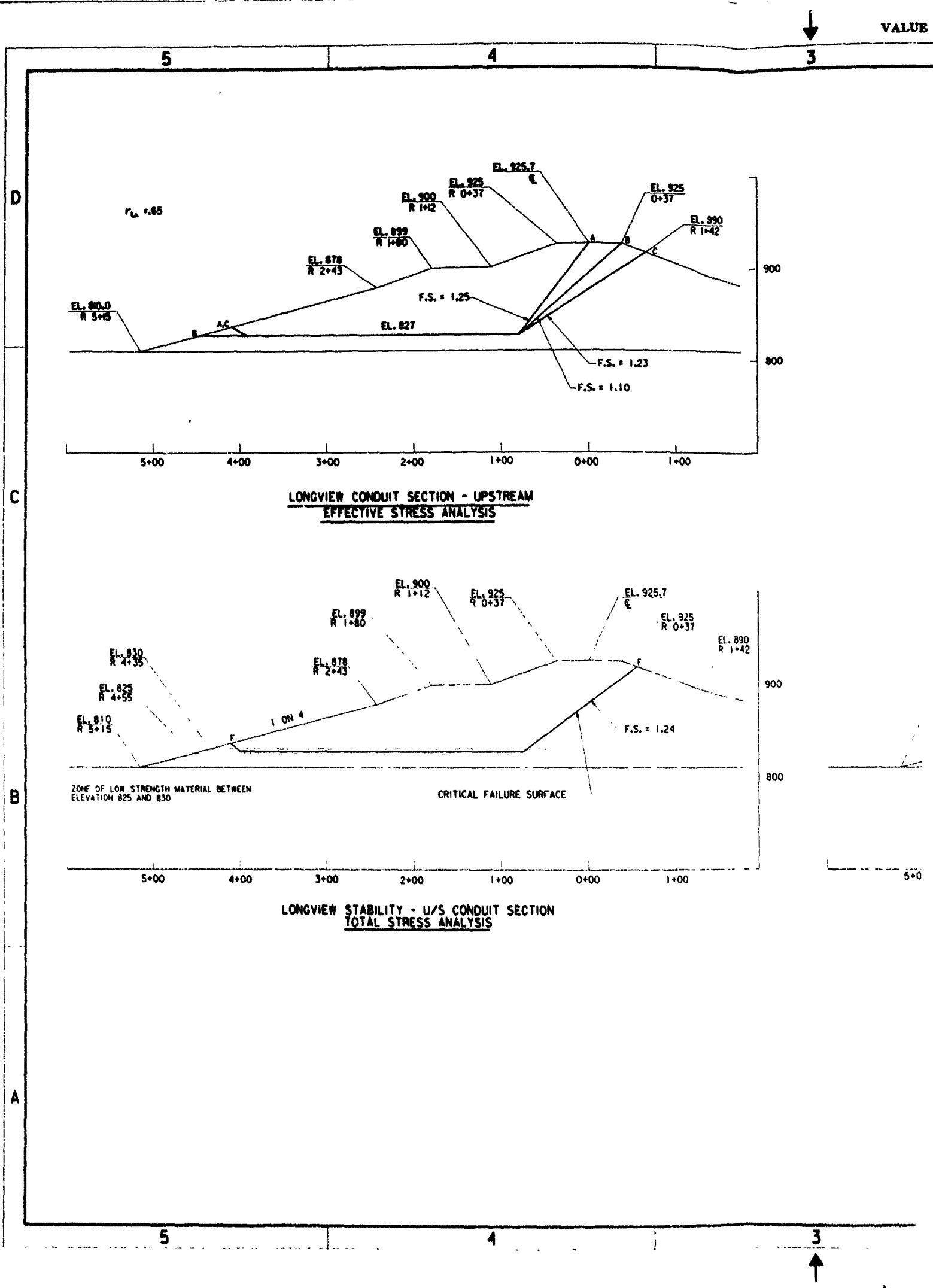
Sheet No. 1

Scale: as shown  
 U. S. ARMY  
 JUNE 1978

Checked by  
 JPM

Recommended  
 Checked by  
 SCD

Approved  
 RBL-3-1568



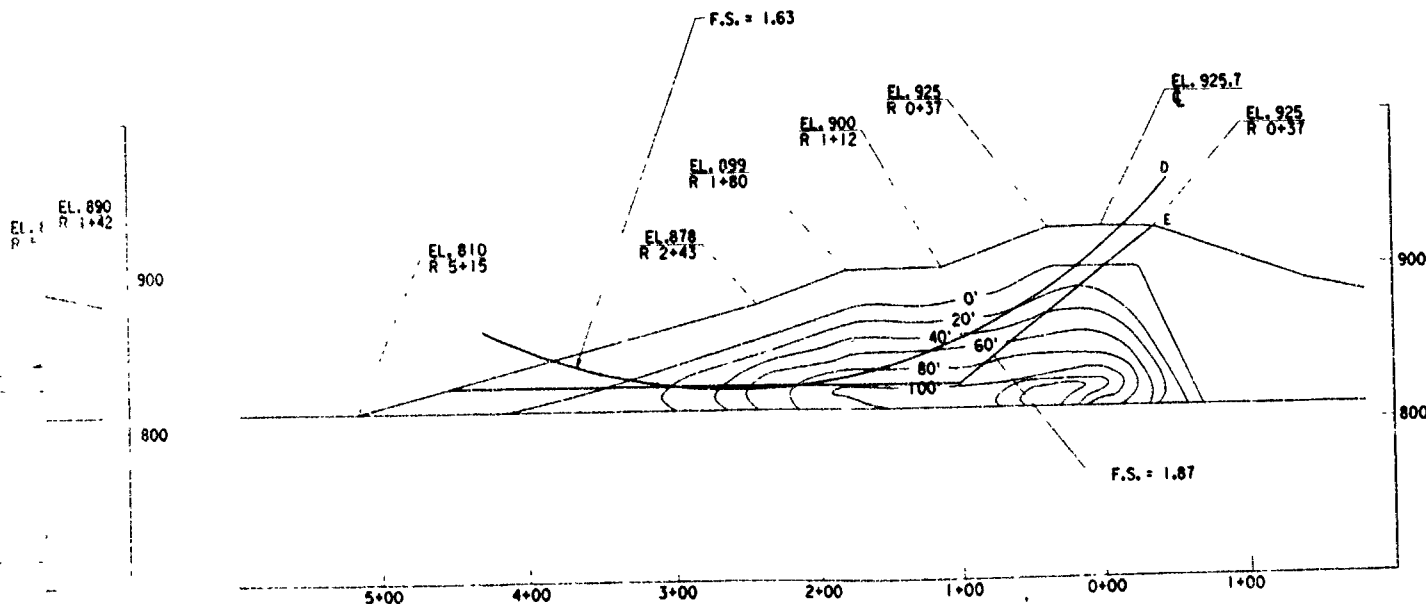
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1

END OF CONSTRUCTION CASE			
SURFACE	FAILURE PLANE EL.	LOCATION	F.S.
A	827	U/S	1.25
B	827	U/S	1.10
C	827	U/S	1.23
D	CIRC. FAILURE SURF.	U/S	1.63
E	827	U/S	1.87
F	827	U/S	1.24

PHYSICAL SOIL CONSTANTS								
MATERIAL	UNIT WEIGHT PCF		DESIGN SHEAR STRENGTH					
			"C"		"R"		"S"	
	SAT.	DRAINED	(KIPS/FT <sup>2</sup> )	TAN Ø	(KIPS/FT <sup>2</sup> )	TAN Ø	(KIPS/FT <sup>2</sup> )	TAN Ø
EMBANKMENT	125.0	115.0	1.80	0.0	.40	.20	0.2	.50
ZONE OF LOW STRENGTH MTRL.	125.0	115.0	1.00	0.0				
SHALE (PEAK)	140.0	140.0	2.0	2.0	2.0	2.00	2.0	2.0



LONGVIEW CONDUIT SECTION - UPSTREAM  
EFFECTIVE STRESS ANALYSIS WITH PORE PRESSURE CONTOURS

Revisions		Date	Approved
Symbol	Descriptions		
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by:	D.A.R.	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT <b>STAGE II EMBANKMENT STABILITY                  ANALYSIS SUMMARY                  CONDUIT SECTION</b>	
Drawn by:	L.M.P.		
Checked by:	L.L.L.		
Submitted by:	R.G.D.		
Scale: 1" = 50'		Sheet Number:	Plot Scale: S=50
Date: X		Design File:	100.55X.05AB
Dwg. No.:		File No.:	RBL-3-1486

							PLACEMENT REQUIREMENTS				
MATERIAL	STAGE	MAJOR SOURCE	MATERIAL DESCRIPTION	CLASS.	ZONE USAGE	EQUIPMENT	MAXIMUM UNCOMPACTED LIFT THICKNESS (INCHES)	NUMBER OF PASSES	DENSITY	MOISTURE	
IMPERVIOUS <sup>o</sup>	I	REQUIRED EXCAVATION AND BORROW	CLAY WITH LL < 65 AND 100 THAT BE CONSISTENTLY HOMOGENEOUS OF 200 AND 400 SIEVE	CL CL	SEPARATION ZONE AND SPECIAL BACKFILL IN SEPARATION AND RANDOM ZONES	TAMPING ROLLER	8	6	% OF MAXIMUM DRY DENSITY AT OPTIMUM MOISTURE CONTENT	+2% TO -2% OF OPTIMUM MOISTURE CONTENT	
IMPERVIOUS <sup>o</sup>	II	REQUIRED EXCAVATION AND STOCKPILE	CLAY WITH 40% CHANNEL UPSTREAM OF 200 SIEVE RELAY CL. NOT LL-65 RELAY CL. NOT BETWEEN 57-60 AND 100-100 SIEVE RELAY CL. NOT BETWEEN 57-60 AND 100-100 SIEVE CUTTING AND RESECTION THICK LL-65 RELAY CL. NOT INCLUDING CUTTING AND RESECTION THICK LL-65 AND 100-100 SIEVE	CL CL	SEPARATION ZONE AND SPECIAL BACKFILL IN SEPARATION AND RANDOM ZONES	TAMPING ROLLER	8	6	% OF MAXIMUM DRY DENSITY AT OPTIMUM MOISTURE CONTENT	+2% TO -2% OF OPTIMUM MOISTURE CONTENT	
RANDOM <sup>o</sup>	I	REQUIRED EXCAVATION AND BORROW	PERVIOUS MATERIAL, EXCEPT 10% OF 200 SIEVE, OR 10% OF 400 SIEVE, OR 10% OF 600 SIEVE, OR 10% OF 800 SIEVE, OR 10% OF 1000 SIEVE, OR 10% OF 1200 SIEVE, OR 10% OF 1400 SIEVE, OR 10% OF 1600 SIEVE, OR 10% OF 1800 SIEVE, OR 10% OF 2000 SIEVE, OR 10% OF 2200 SIEVE, OR 10% OF 2400 SIEVE, OR 10% OF 2600 SIEVE, OR 10% OF 2800 SIEVE, OR 10% OF 3000 SIEVE, OR 10% OF 3200 SIEVE, OR 10% OF 3400 SIEVE, OR 10% OF 3600 SIEVE, OR 10% OF 3800 SIEVE, OR 10% OF 4000 SIEVE, OR 10% OF 4200 SIEVE, OR 10% OF 4400 SIEVE, OR 10% OF 4600 SIEVE, OR 10% OF 4800 SIEVE, OR 10% OF 5000 SIEVE, OR 10% OF 5200 SIEVE, OR 10% OF 5400 SIEVE, OR 10% OF 5600 SIEVE, OR 10% OF 5800 SIEVE, OR 10% OF 6000 SIEVE, OR 10% OF 6200 SIEVE, OR 10% OF 6400 SIEVE, OR 10% OF 6600 SIEVE, OR 10% OF 6800 SIEVE, OR 10% OF 7000 SIEVE, OR 10% OF 7200 SIEVE, OR 10% OF 7400 SIEVE, OR 10% OF 7600 SIEVE, OR 10% OF 7800 SIEVE, OR 10% OF 8000 SIEVE, OR 10% OF 8200 SIEVE, OR 10% OF 8400 SIEVE, OR 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14								



[illegible]

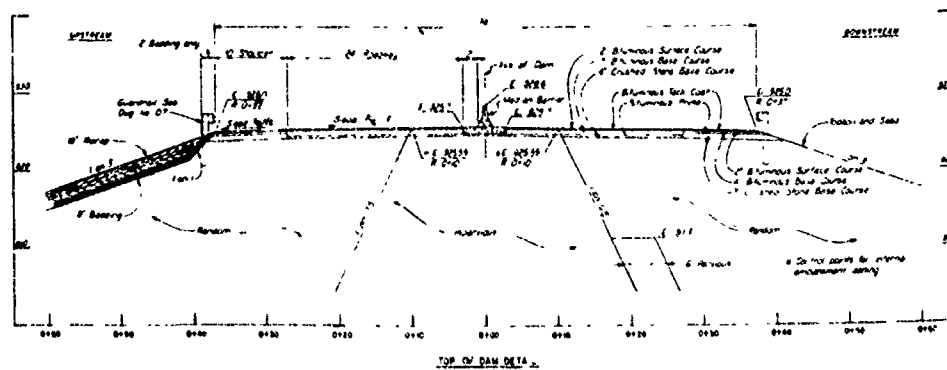
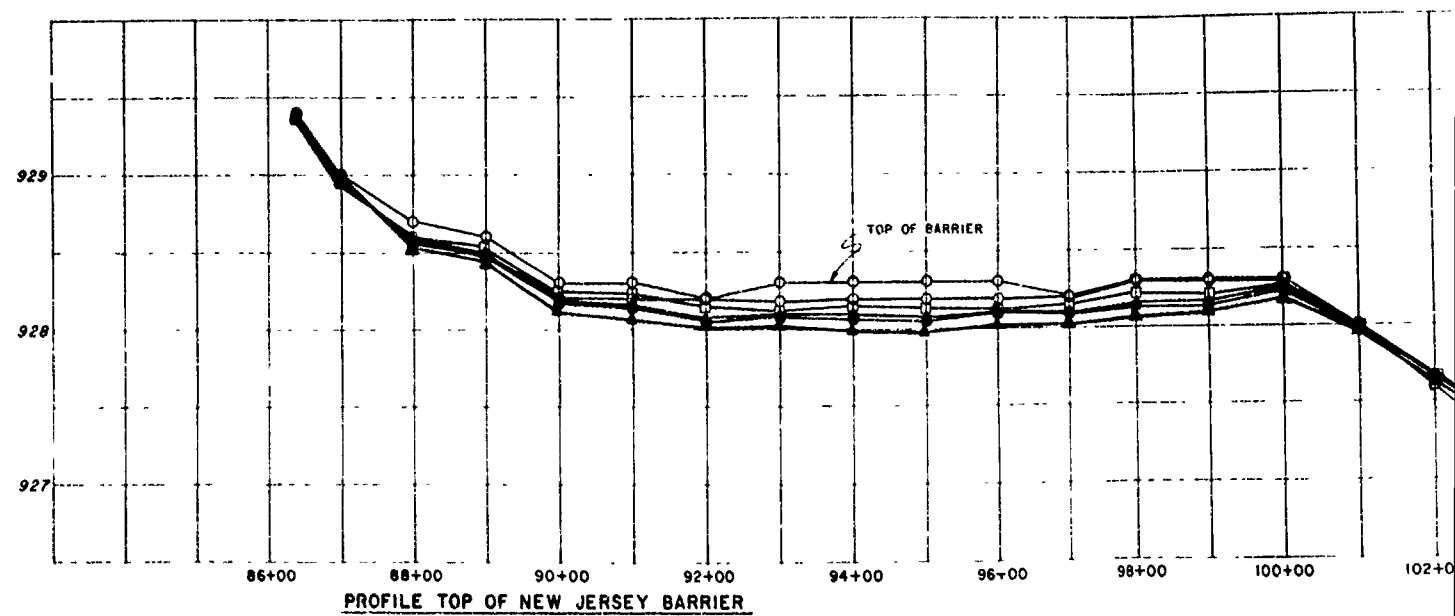
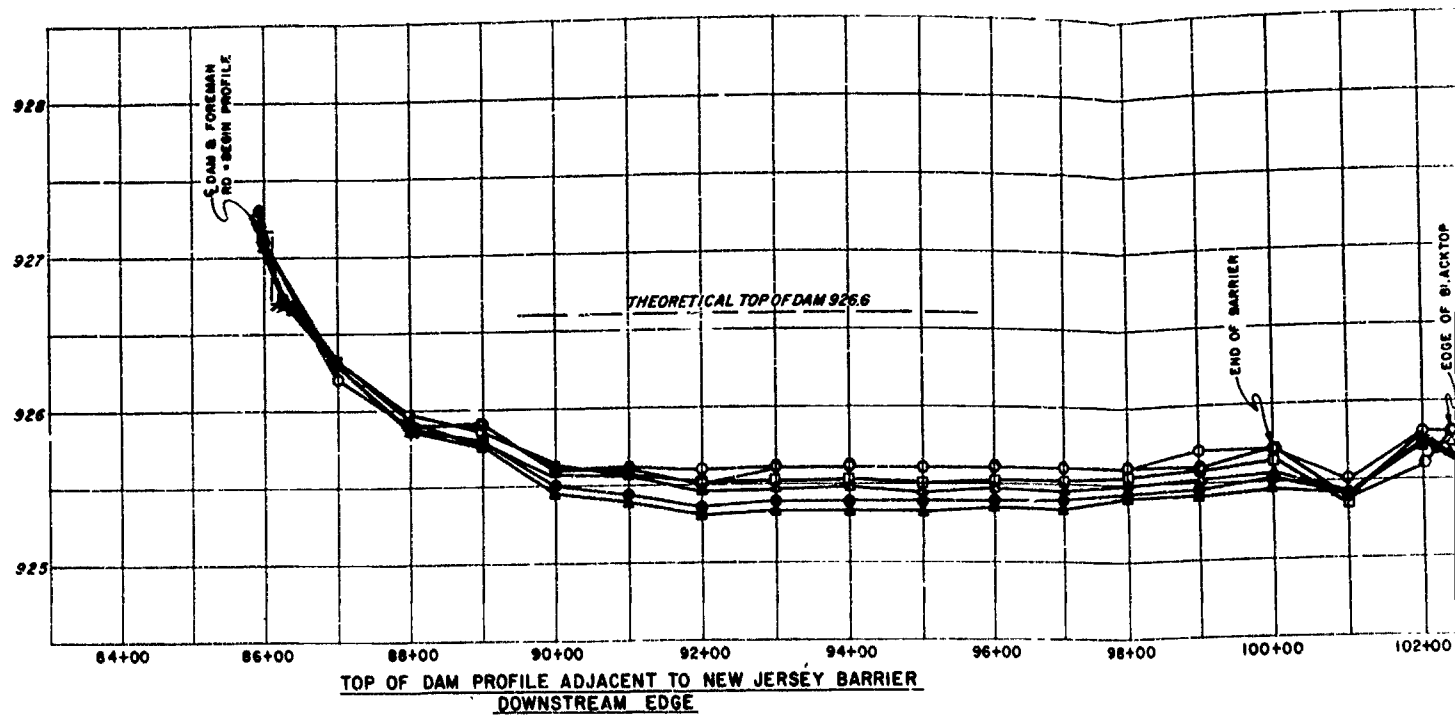
MATERIAL	STAGE I (YD <sup>3</sup> )	STAGE II (YD <sup>3</sup> )	TOTAL (YD <sup>3</sup> )
PERVIOUS	24,387	74,197	104,584
IMPERVIOUS	992,886	918,662	1,626,508
ARMOR	215,958	898,361	914,313
SEDG	246,467	621,354	908,871

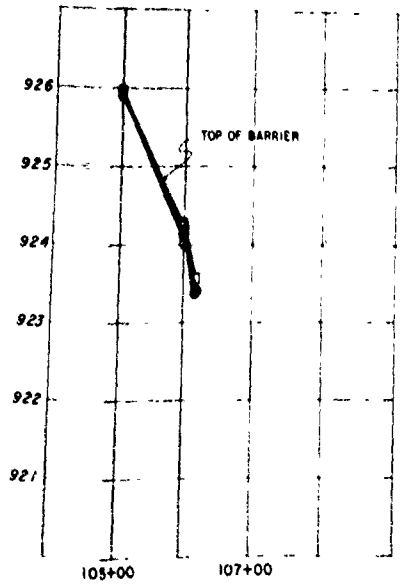
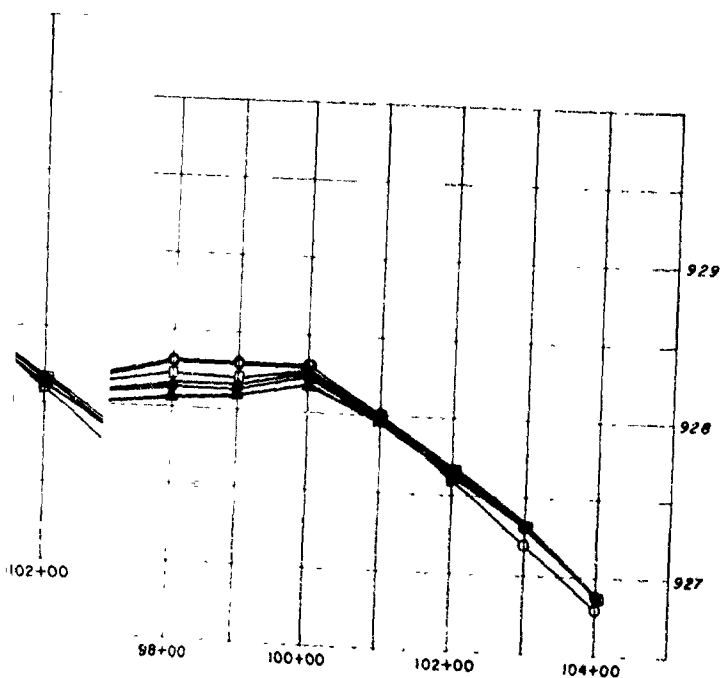
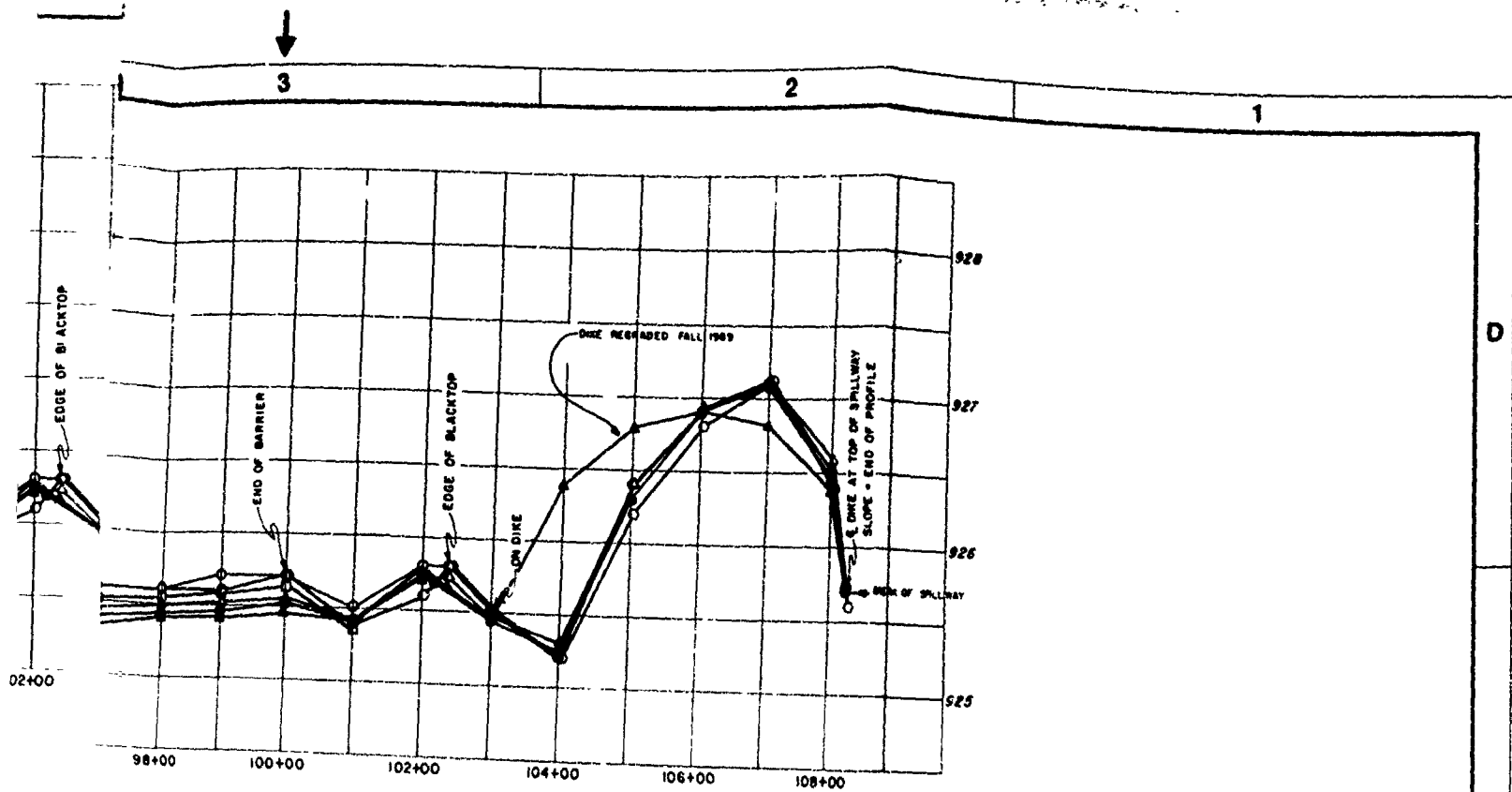
\* WHEN PLACED OVER A ROCK FOUNDATION, THE LIFT THICKNESS WAS 6 INCHES AND COMPACTION WAS ACHIEVED WITH TWO PASSES OF A RUBBER-TYRED ROLLER UNTIL THE FILL REACHED 18 INCHES THICKNESS OVER THE ROCK FOUNDATION. THE RUBBER-TYRED ROLLER CAUSED BREAKAGE OF SHALE FOUNDATION. THE BALLAST WAS LESSENED ON OTHER ROLLERS WERE USED AS APPROVED BY THE CONTRACTING OFFICER.

\*70% RELATIVE DENSITY WAS NOT ACHIEVABLE AND THE SPECIFIED DENSITY WAS REDUCED TO 70% RELATIVE DENSITY.

[illegible]

ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1929



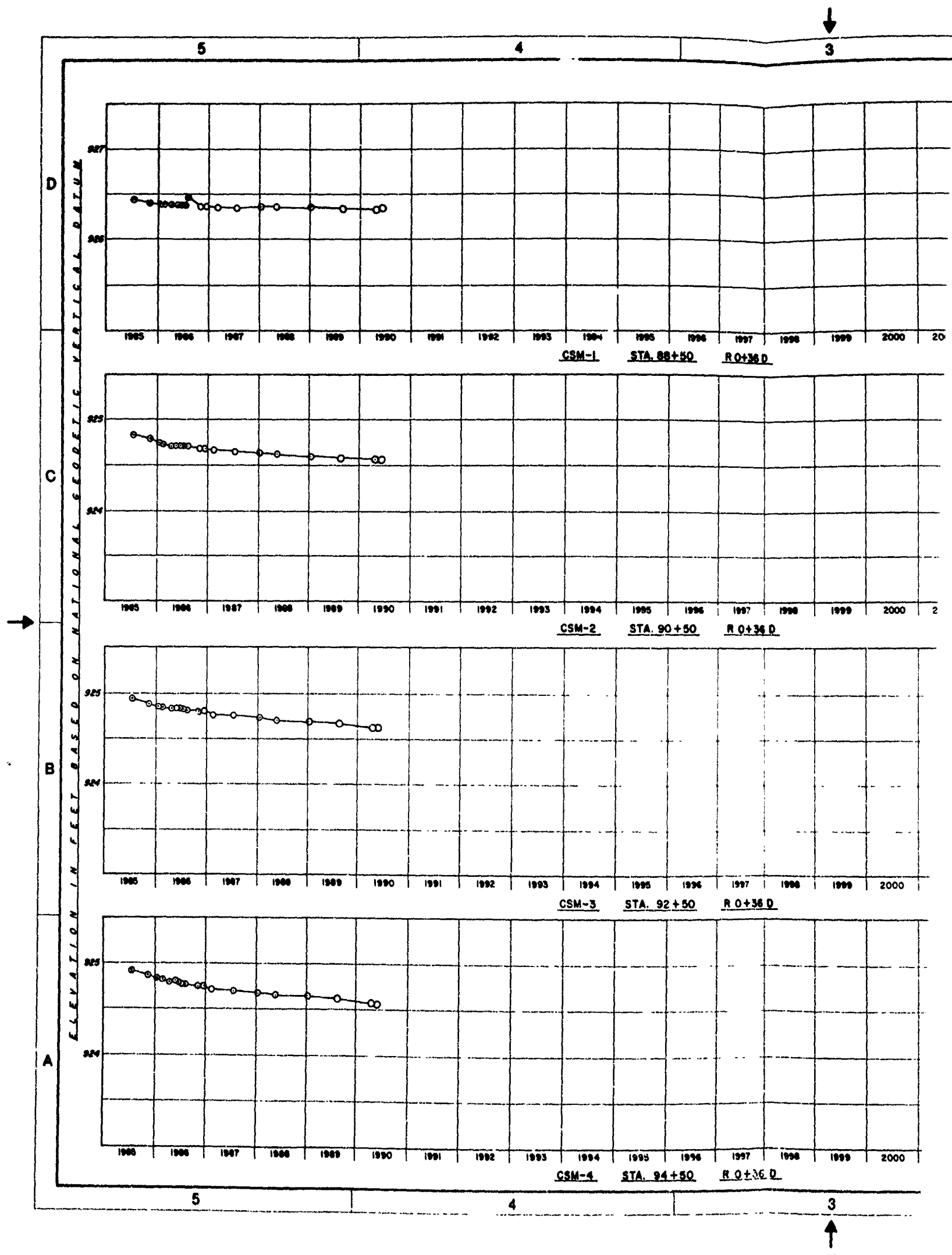


# LEGEND

FILE 8 NOV '85  
5 MAR '87  
4 JUN '87  
13 MAR '88  
12 JAN '89  
23 APR '90

LEGEND	
INITIAL PROFILE 8 NOV '85	○
5 MAR '87	○
4 JUN '87	○
13 MAR '88	△
12 JAN '89	●
23 APR '90	▲

Revisions			
Symbol	Descriptions	Date	Approved
	Revised March 1990		
U S ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by RGO	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT		
Drawn by VAB	TOP OF DAM PROFILE PLOTS		
Checked by RLA	Scale AS SHOWN	Sheet number	
Submitted by WBE	Date DECEMBER 1986		
	Draw No	File No	RBL-3-1570





3

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927

926

2001 1997 1998 1999 2000 2001 2002 2003 2004 2005 2006

0+36.0

925

924

2001 1997 1998 1999 2000 2001 2002 2003 2004 2005 2006

0+36.0

925

924

2001 1997 1998 1999 2000 2001 2002 2003 2004 2005 2006

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925

924

2001 1997 1998 1999 2000 2001 2002 2003 2004 2005 2006

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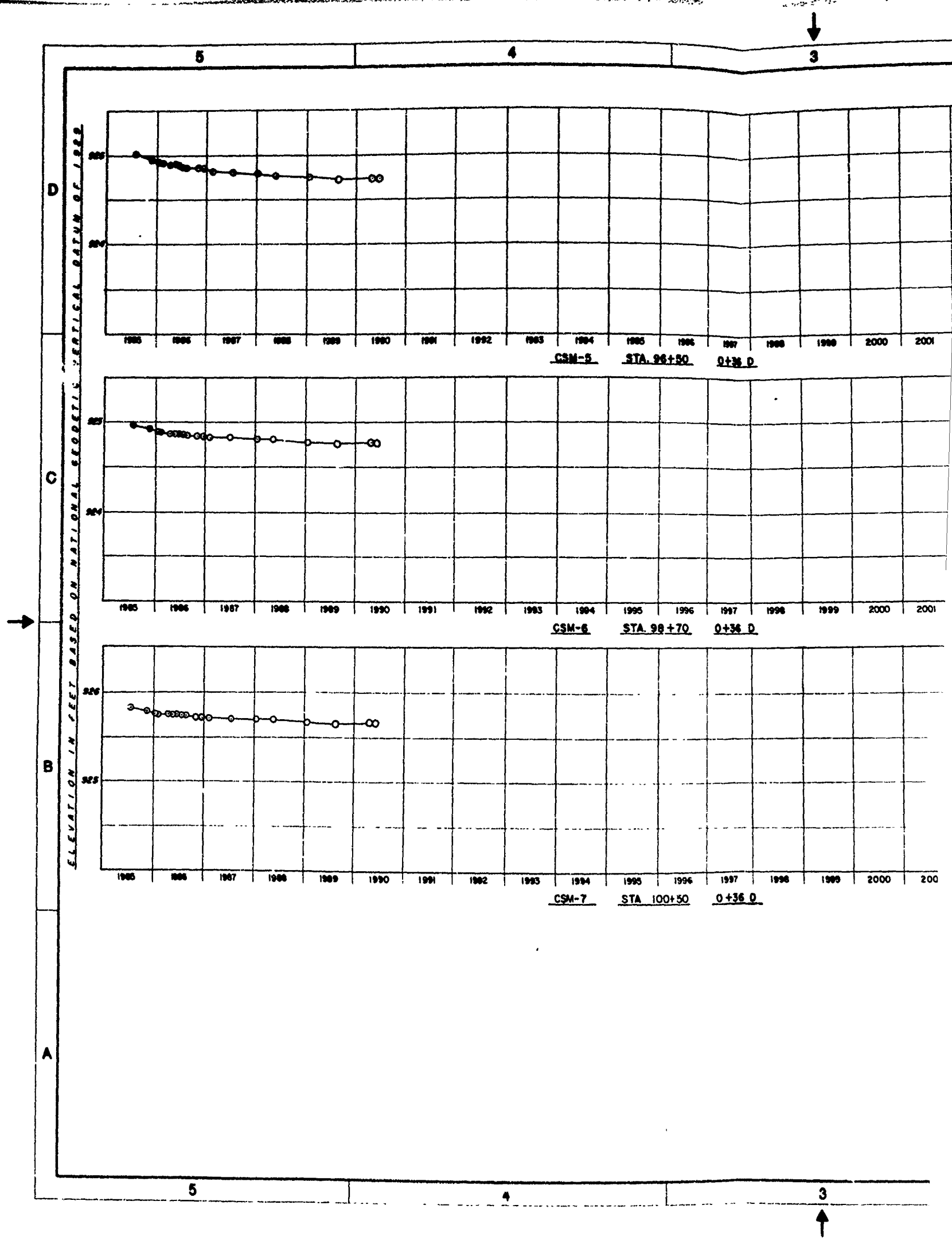
Revisions			
Symbol	Descriptions	Date	Approved
	Revised March 1990		
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by RCD	<div>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT  <b>CREST SETTLEMENT MONUMENT PLOTS</b></div>		
Drawn by EGM			
Checked by RLA			
Submitted by WBE			
Scale: AS SHOWN		Sheet number.	
Date: DECEMBER 1986			
Draw No: 1 OF 2		File No: RBL-3-1571	

PLATE NO. 95



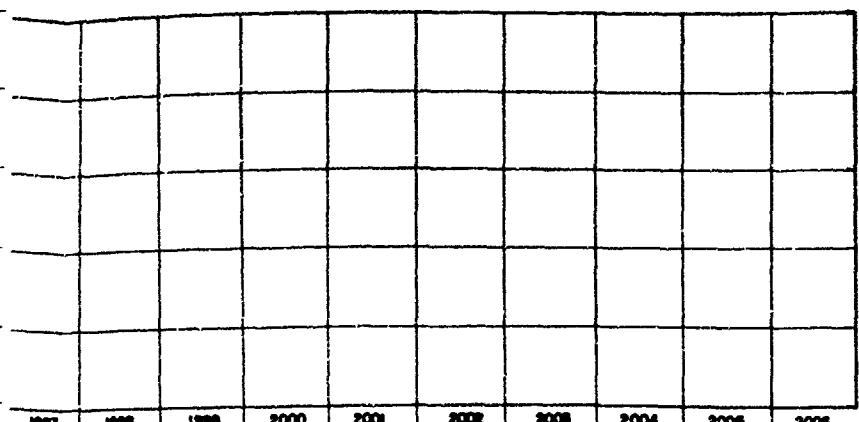


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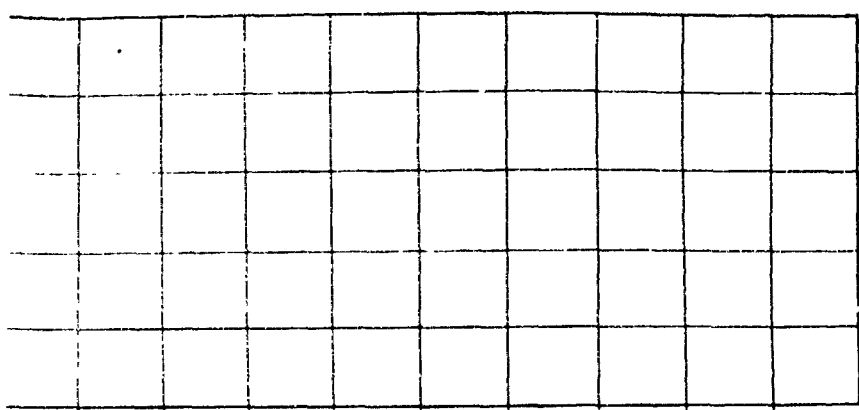
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1997 1998 1999 2000 2001 2002 2003 2004 2005 2006

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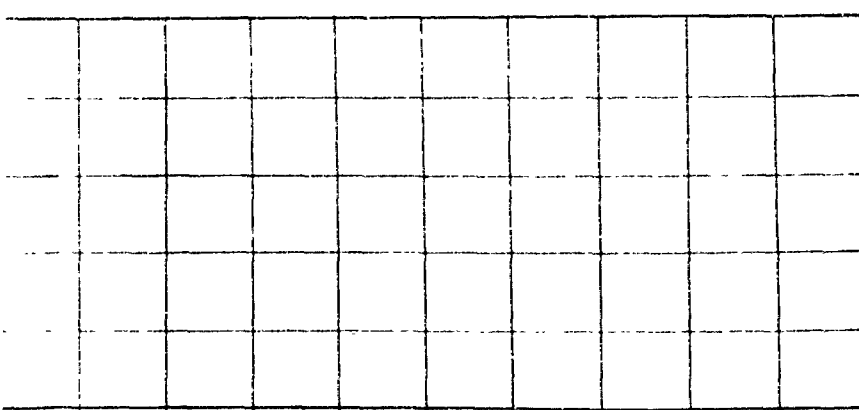
2001



1997 1998 1999 2000 2001 2002 2003 2004 2005 2006

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1997 1998 1999 2000 2001 2002 2003 2004 2005 2006

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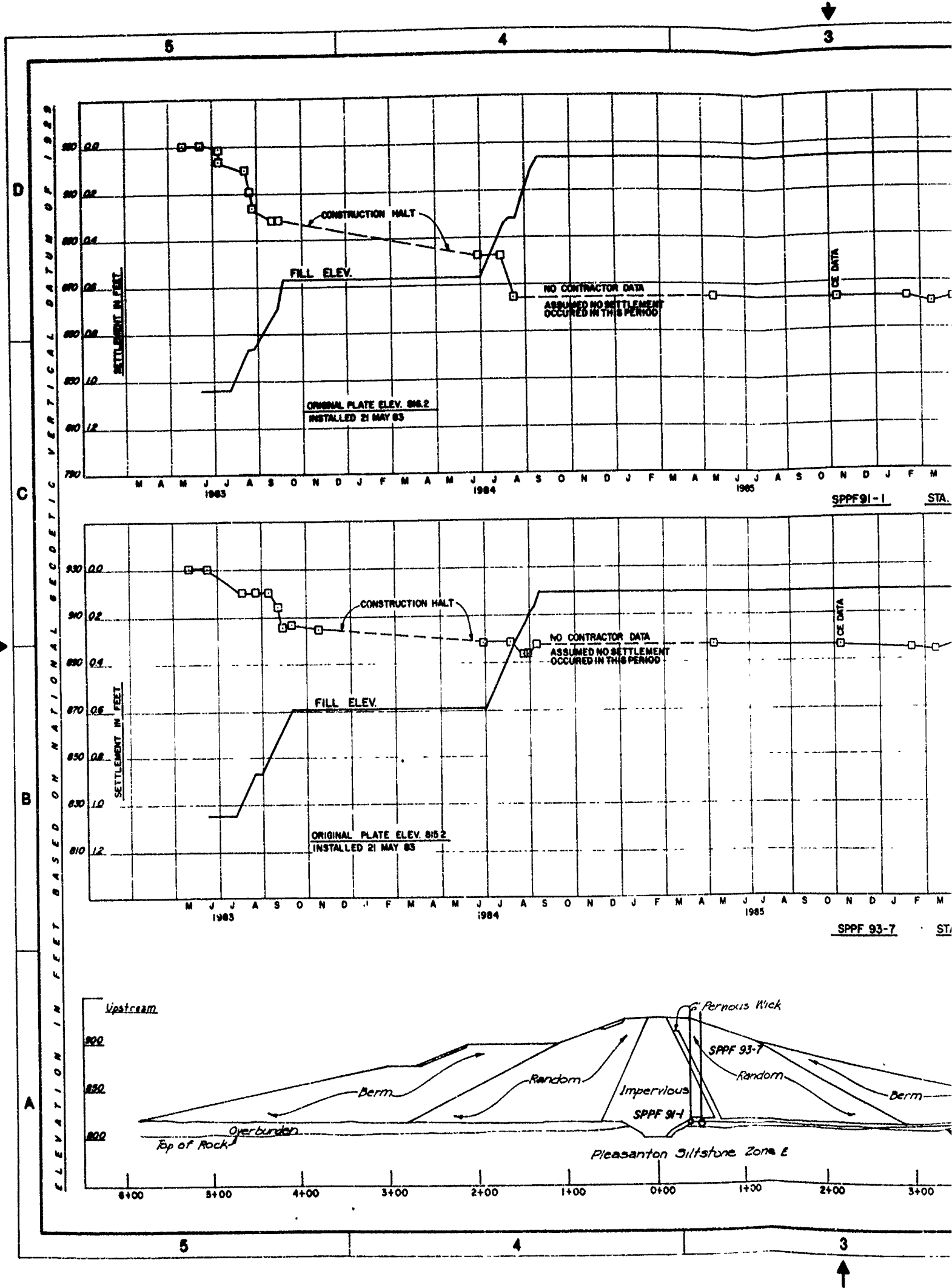
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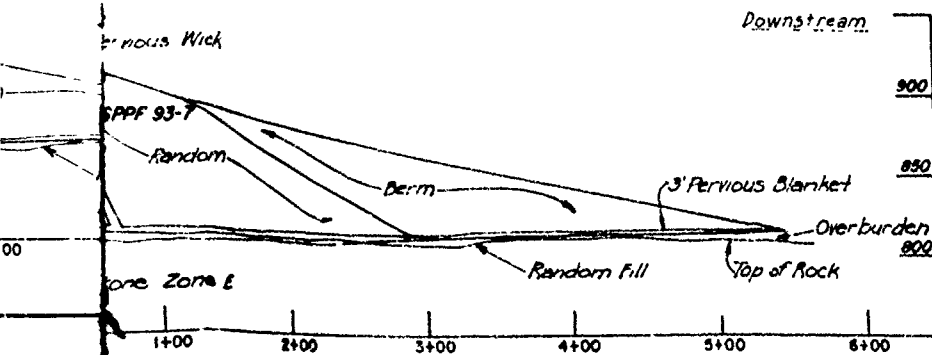
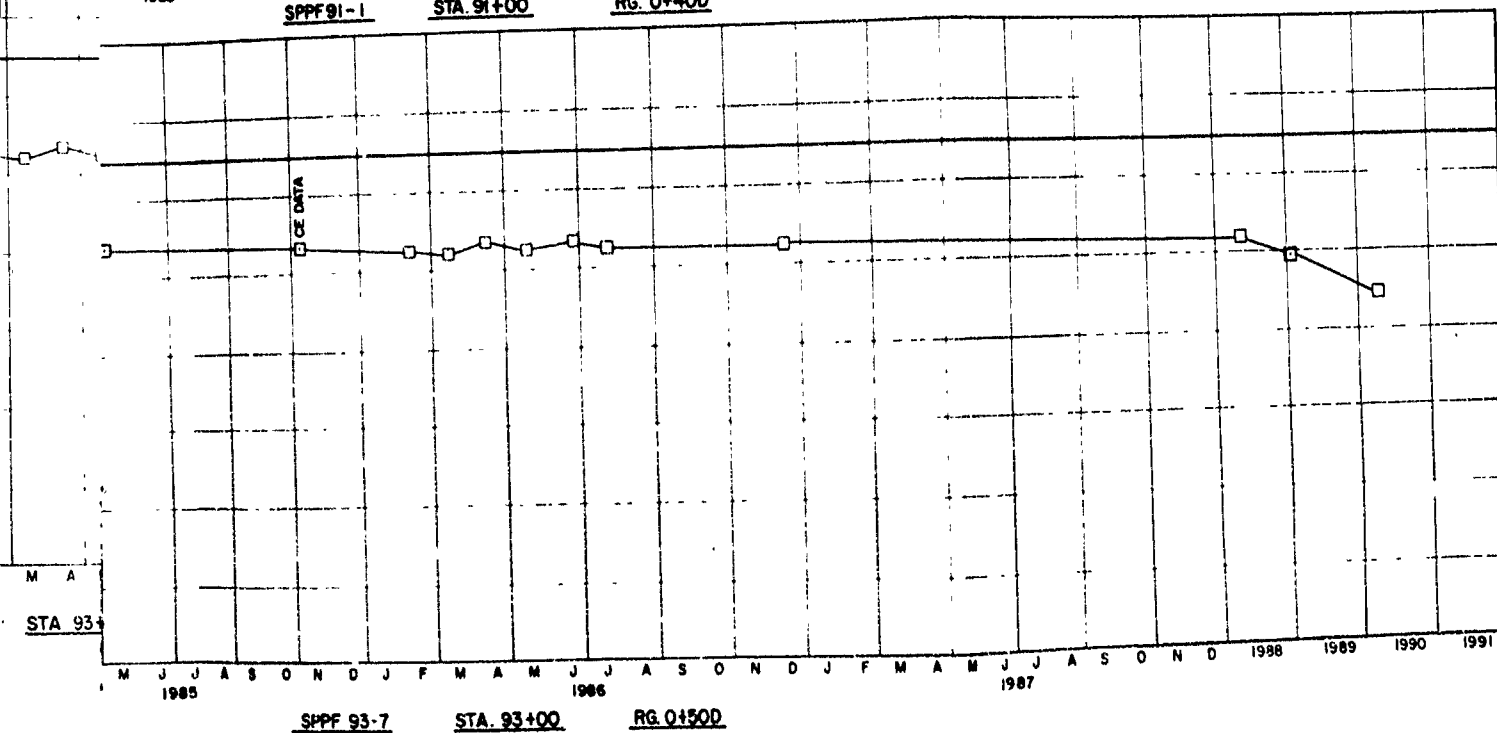
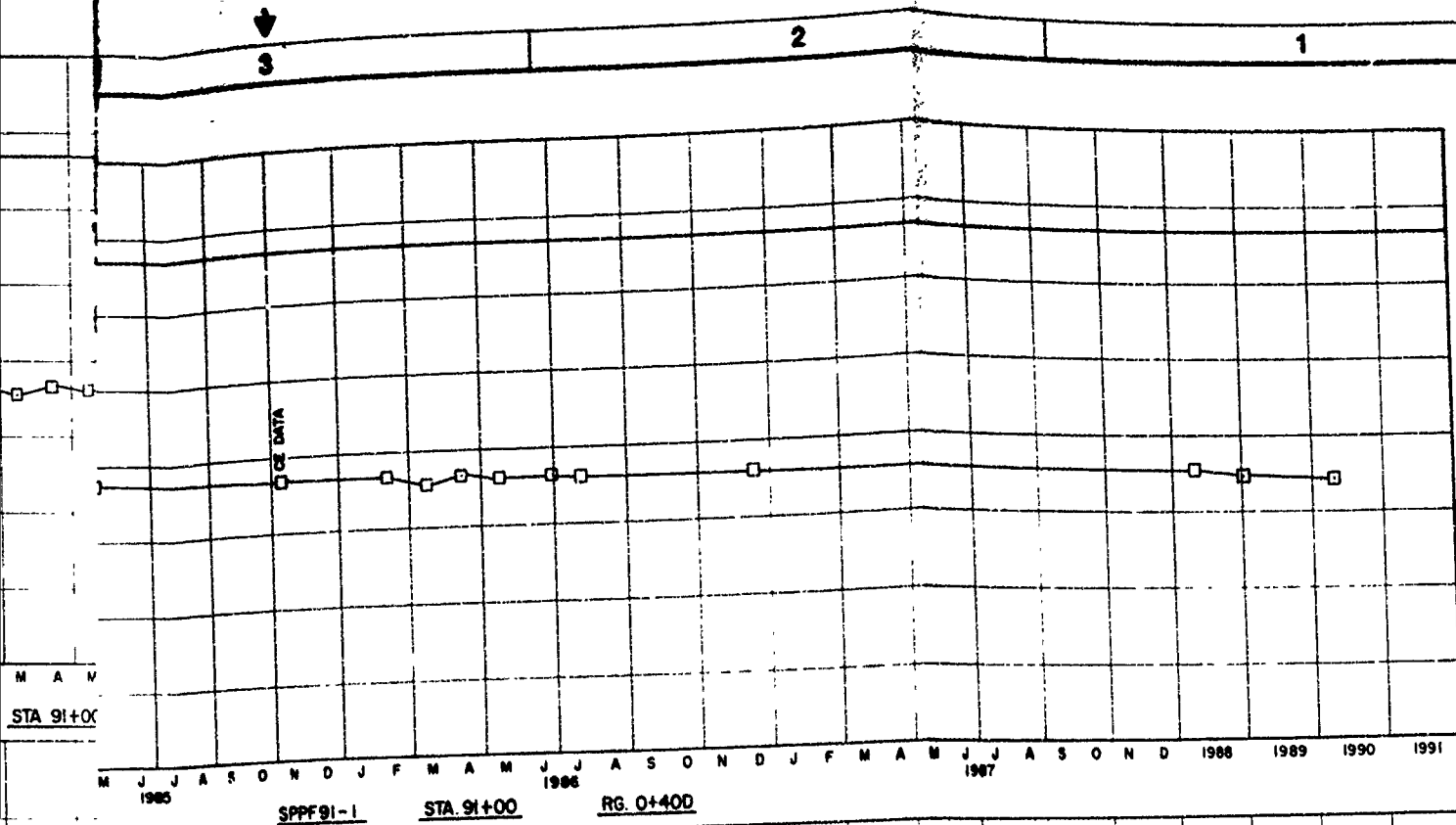
PLATE NO. 96

Revisions			
Symbol	Descriptions	Date	Approved
	Revised March 1990		
U. S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by: R G D		LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by: J G S		CREST SETTLEMENT MONUMENT PLOTS	
Checked by: R L A		Scale: AS SHOWN	Sheet number
Submitted by: W B E		Date: DECEMBER 1986	File No. RBL-3-1572
Drawn by: J G S		Date: DECEMBER 1986	File No. RBL-3-1572
Checked by: R L A		Date: DECEMBER 1986	File No. RBL-3-1572
Submitted by: W B E		Date: DECEMBER 1986	File No. RBL-3-1572

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Revisions		Date	Approved
Symbol	Description		
	Revised March 1990		

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT

Settlement Plate Data Plots  
SPPF 91-1 and SPPF 93-7

Designed by R G D	Scale AS SHOWN	Sheet number 1
Drawn by S E Y	Date DECEMBER 1988	
Checked by R L A	Drawn by W B E	File No RBL-3-1573

PLATE NO. 97

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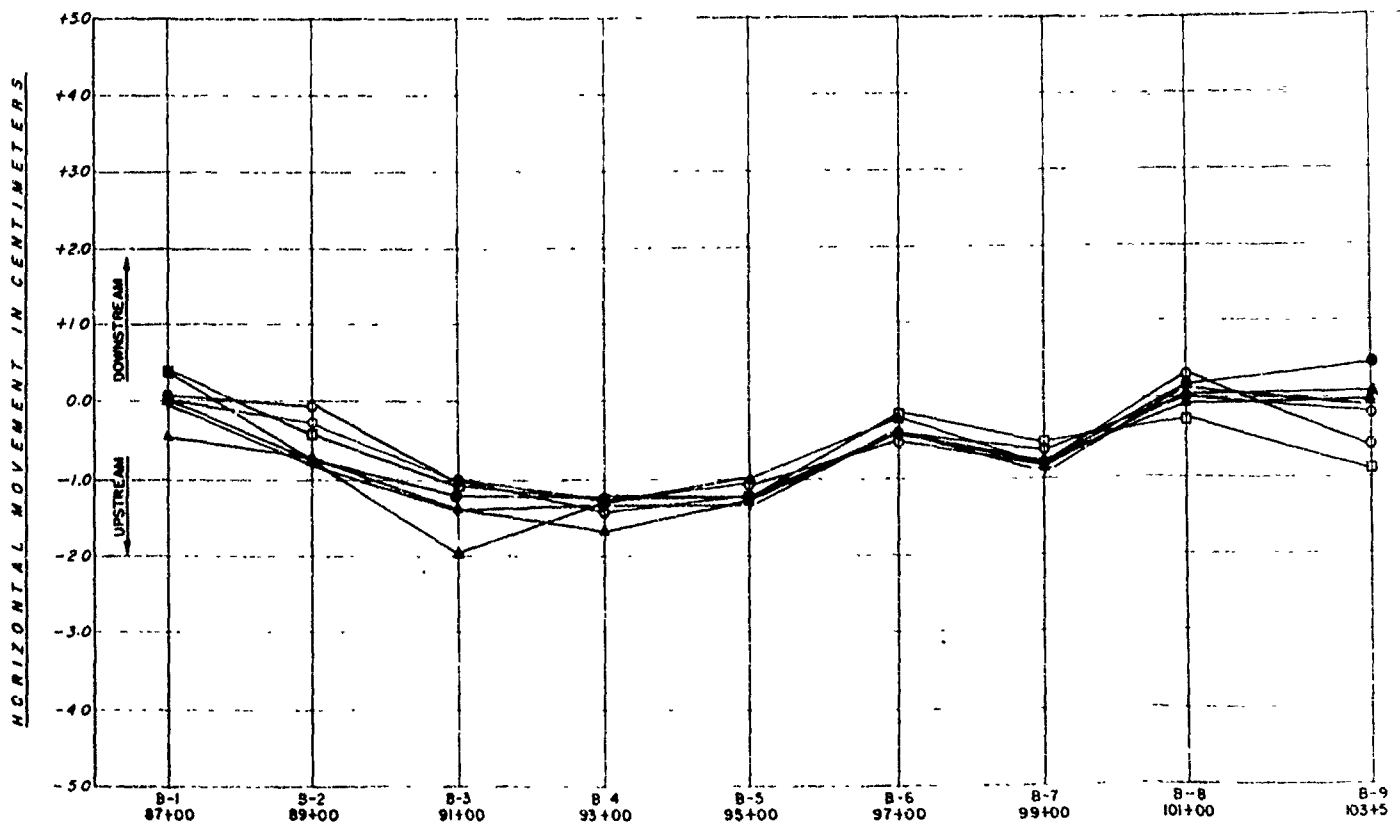
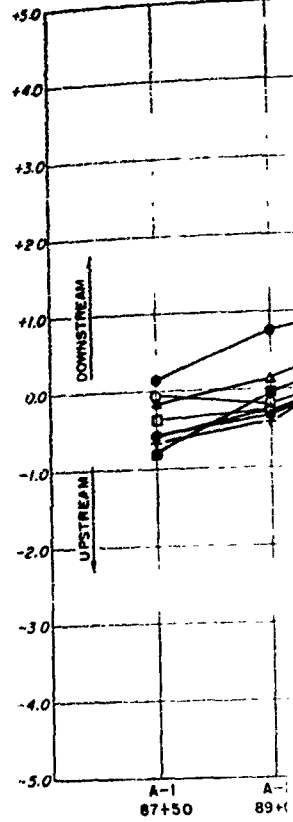
**LEGEND**

ORIGINAL MEANED SURVEY	
3 SEPT 87	○
7 JAN 88	□
13 APR 88	○
11 JAN 89	△
9 AUG 89	●
23 APR 90	▲
17 MAY 90	■
1 JUN 90	+

**ORIGINAL MEANED SURVEY DATES**

LINE "A"	LINE "B"
26 JULY 1985	20 AUG 1985
20 AUG 1985	30 SEPT 1985 <sup>1</sup>
30 SEPT 1985 <sup>1</sup>	30 SEPT 1985 <sup>2</sup>

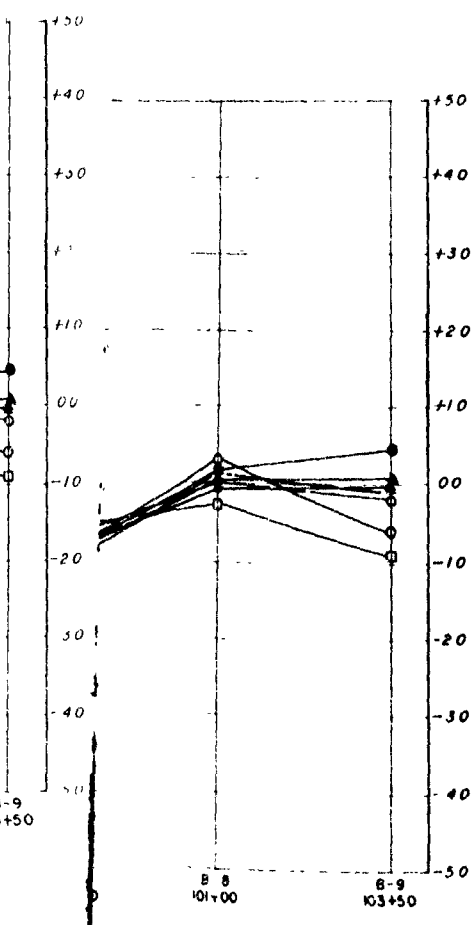
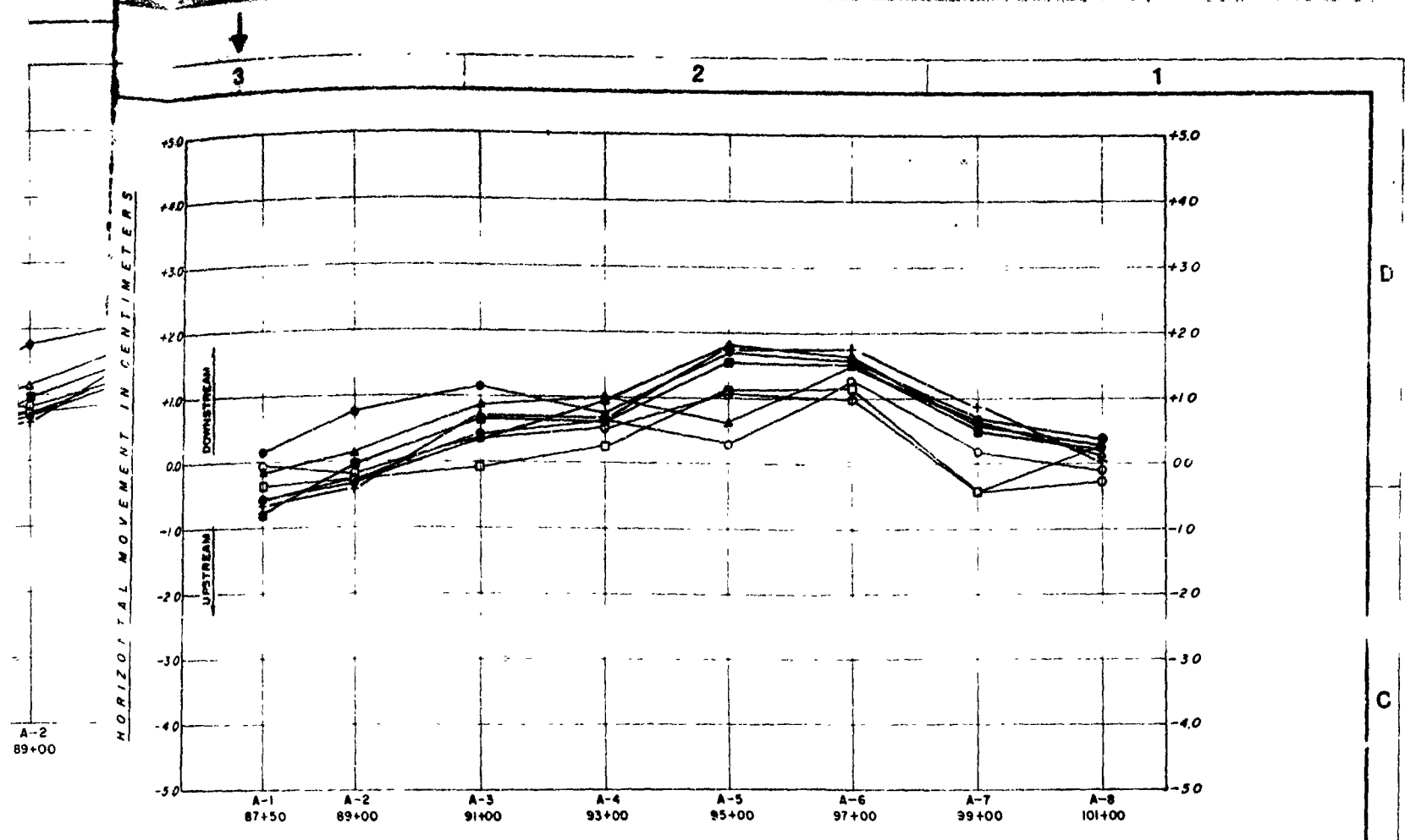
HORIZONTAL MOVEMENT IN CENTIMETERS



ALIGNMENT LINE "B" MONUMENTS  
RANGE 1+70 UPSTREAM

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21.11



Symbol	Revisions	Date	Approved
	Revised March 1990		

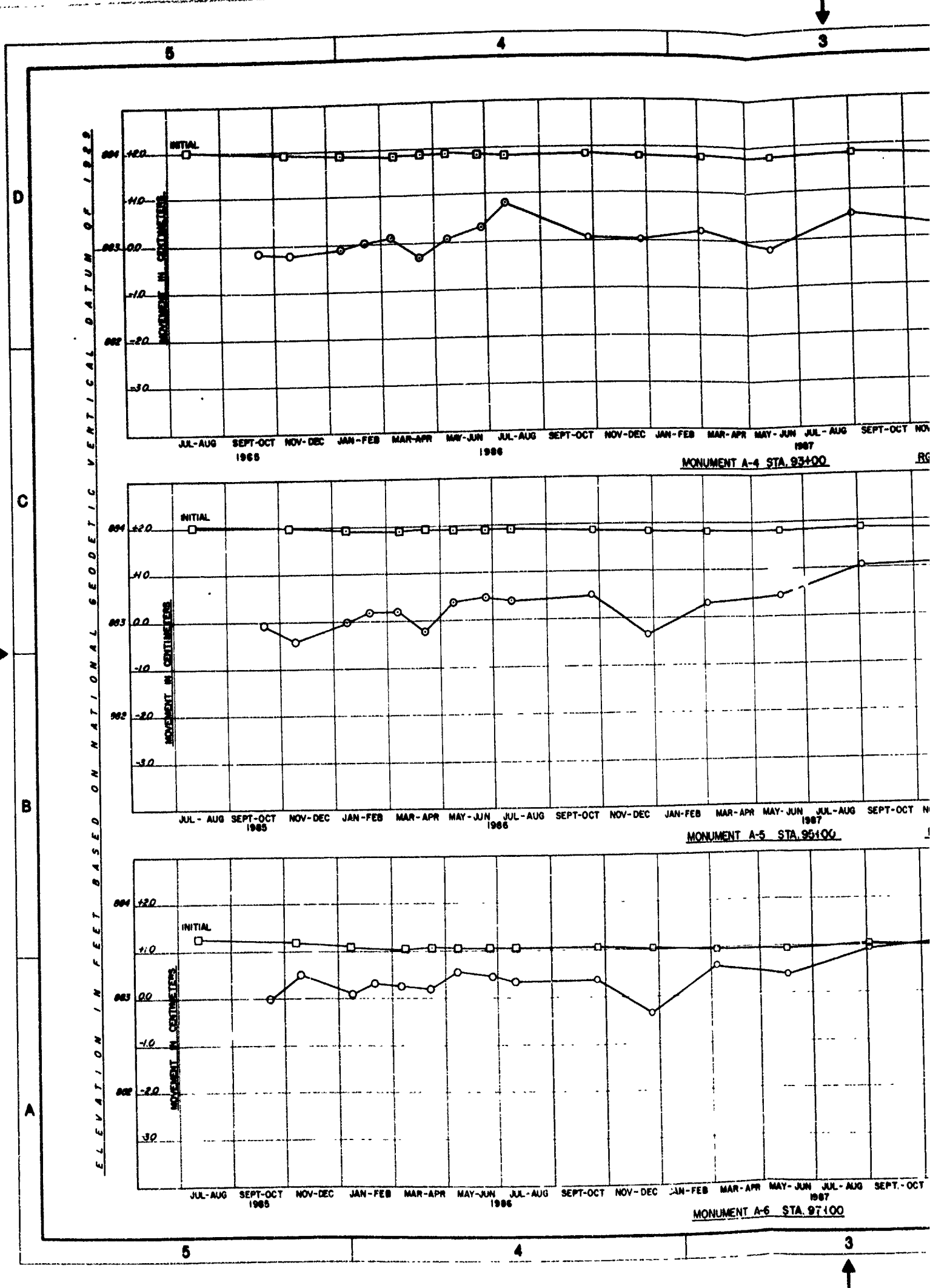
U. S. ARMY ENGINEER DISTRICT  
 CORPS OF ENGINEERS  
 KANSAS CITY, MISSOURI

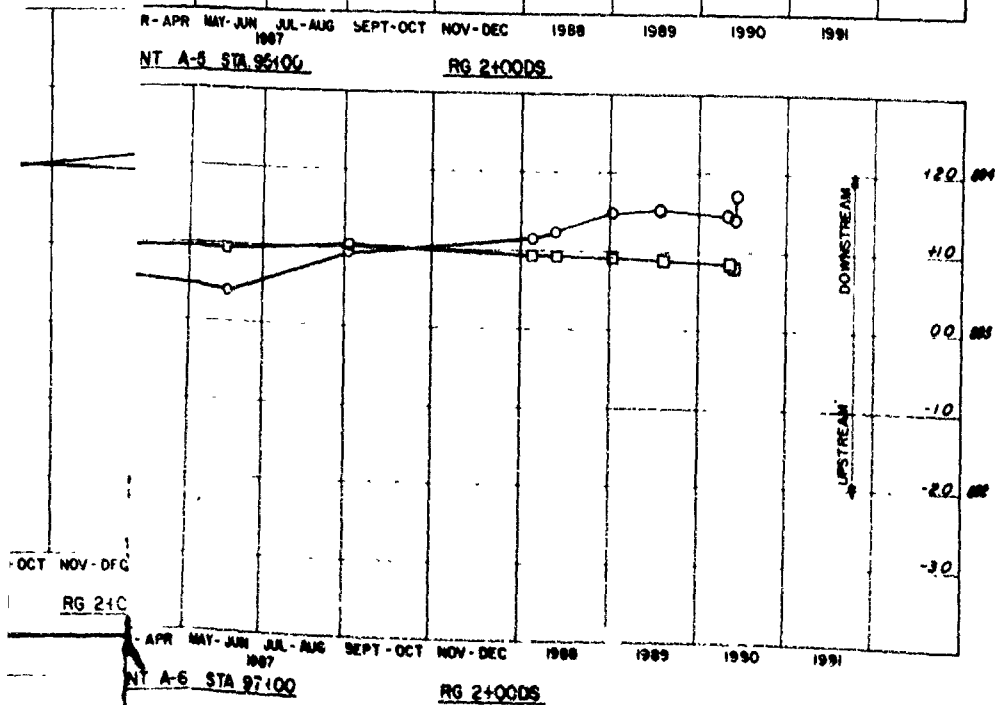
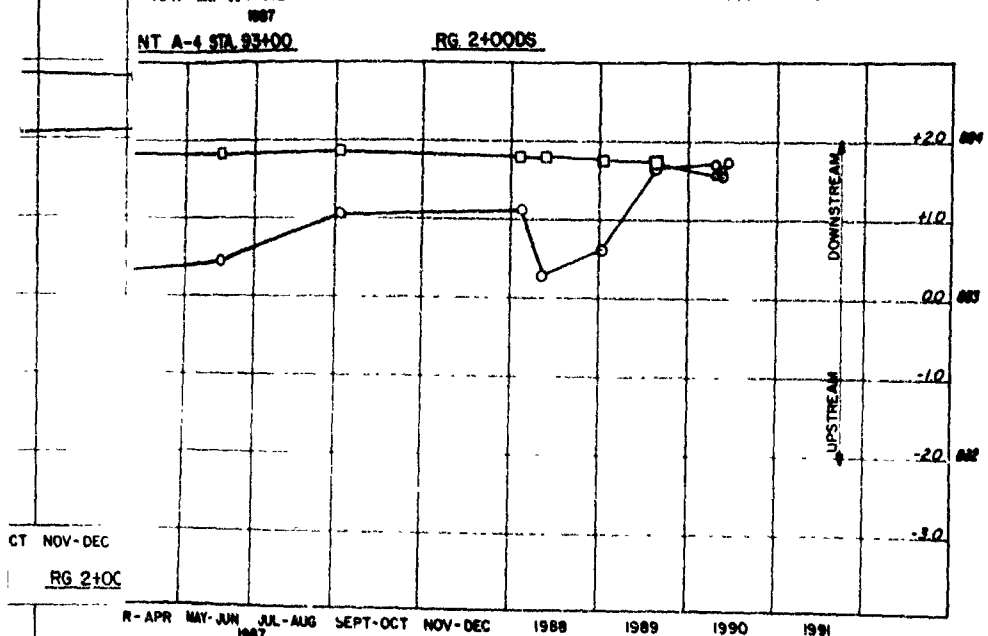
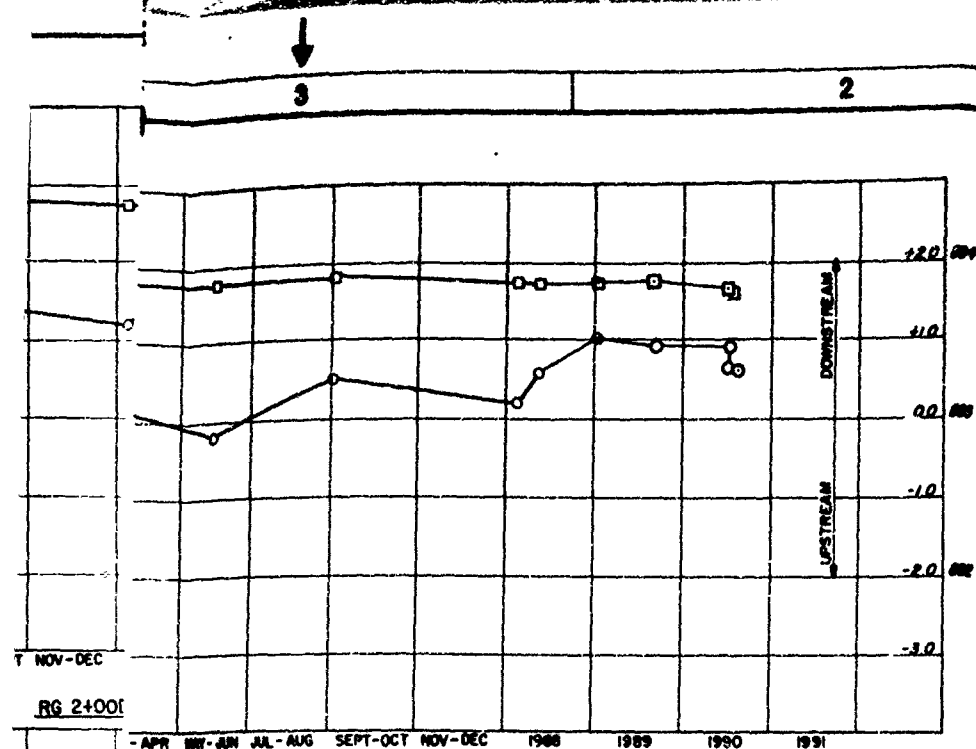
DESIGNED BY: RGD  
 DRAWN BY: RDT  
 CHECKED BY: RLA  
 SUBMITTED BY: WBE

**LITTLE BLUE RIVER, MISSOURI  
 LONGVIEW LAKE  
 EMBANKMENT CRITERIA AND  
 PERFORMANCE REPORT**

**ALIGNMENT MONUMENT HORIZONTAL PROFILE  
 ORIGINAL SURVEY BASE LINES  
 LINES "A" AND "B" SUMMARY**

Scale: AS SHOWN  
 Date:   
 Draw No: 4 of 4  
 Sheet number:   
 File No: RBL-3-1574





**LEGEND**  
 VERTICAL MOVEMENT ----- □  
 HORIZONTAL MOVEMENT ----- ○

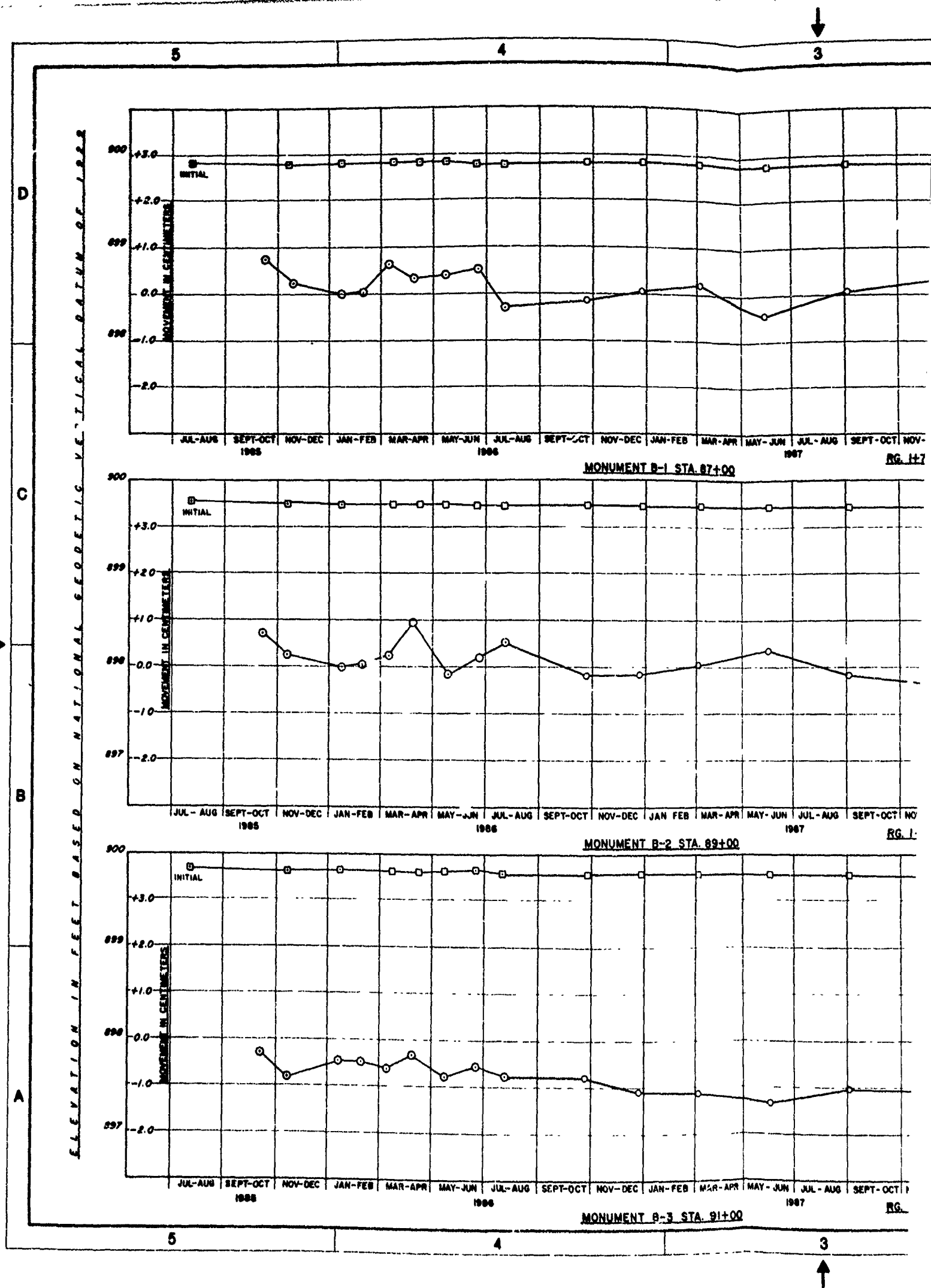
Revisions			
Symbol	Description	Date	Approved
	Revised March 1990		

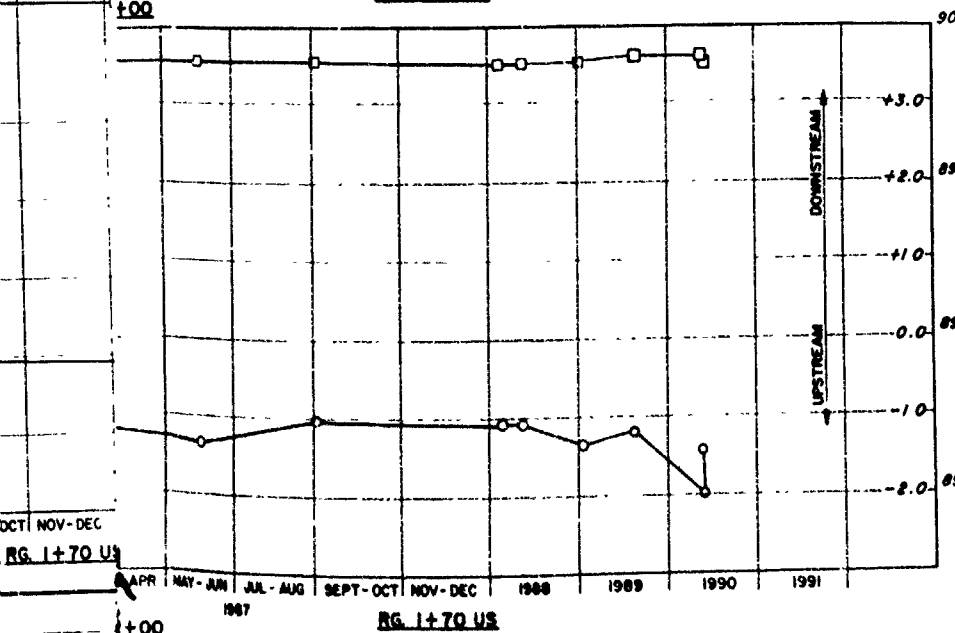
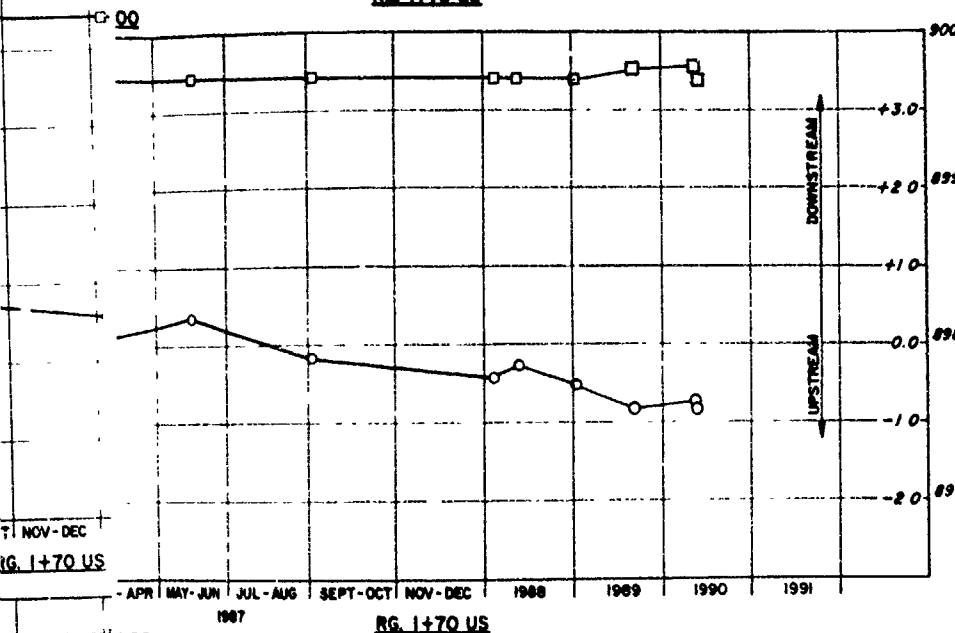
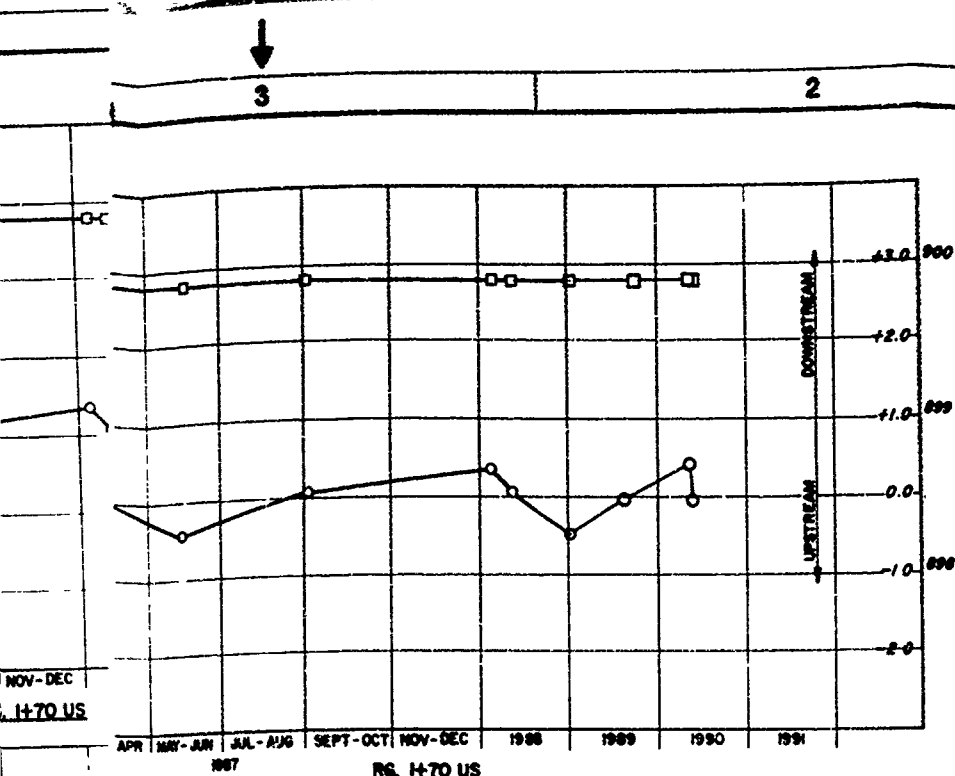
U. S. ARMY ENGINEER DISTRICT  
 CORPS OF ENGINEERS  
 KANSAS CITY, MISSOURI

Designed by: RGD  
 Drawn by: SEY  
 Checked by: RLA  
 Submitted by: WSE

LITTLE BLUE RIVER, MISSOURI  
 LONGVIEW LAKE  
 EMBANKMENT CRITERIA AND  
 PERFORMANCE REPORT  
 ALIGNMENT MONUMENTS HORIZONTAL AND  
 VERTICAL DATA PLOTS MONUMENTS A-4,  
 A-5 AND A-6.

Scale: AS SHOWN  
 Date: DECEMBER 1986  
 Sheet number:   
 File No: RBL-3-1575





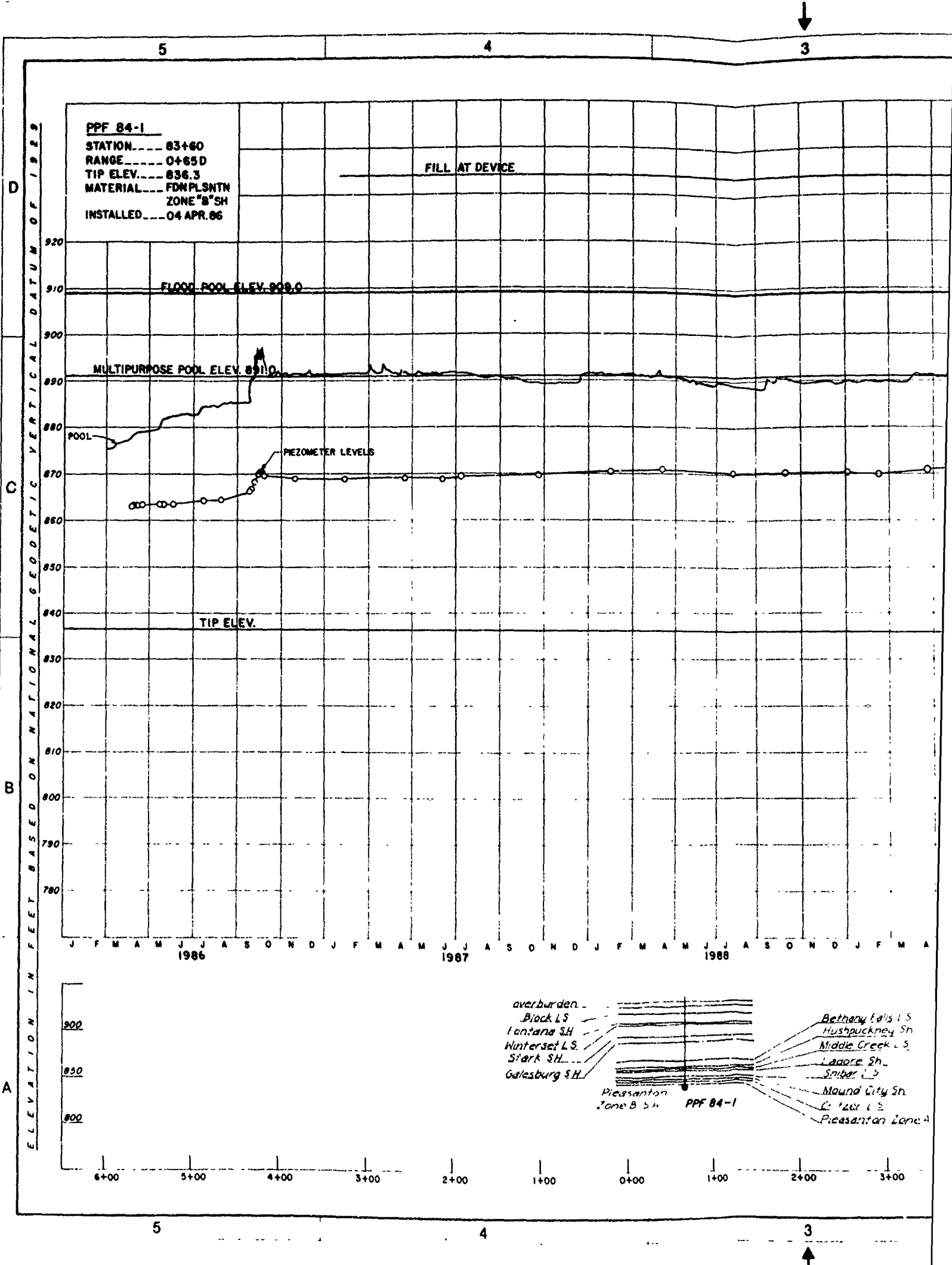
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 HORIZONTAL MOVEMENT ——— ○

Revisions			
Symbol	Descriptions	Date	Approved
	Revised March 1990		

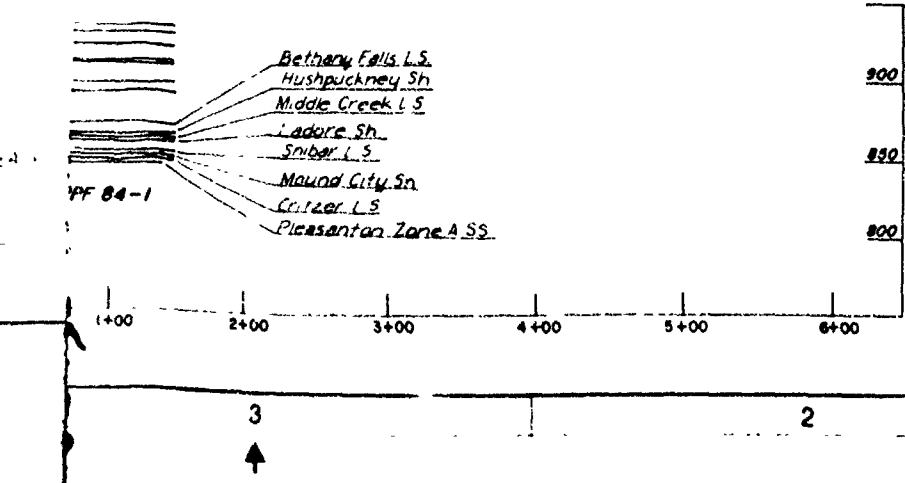
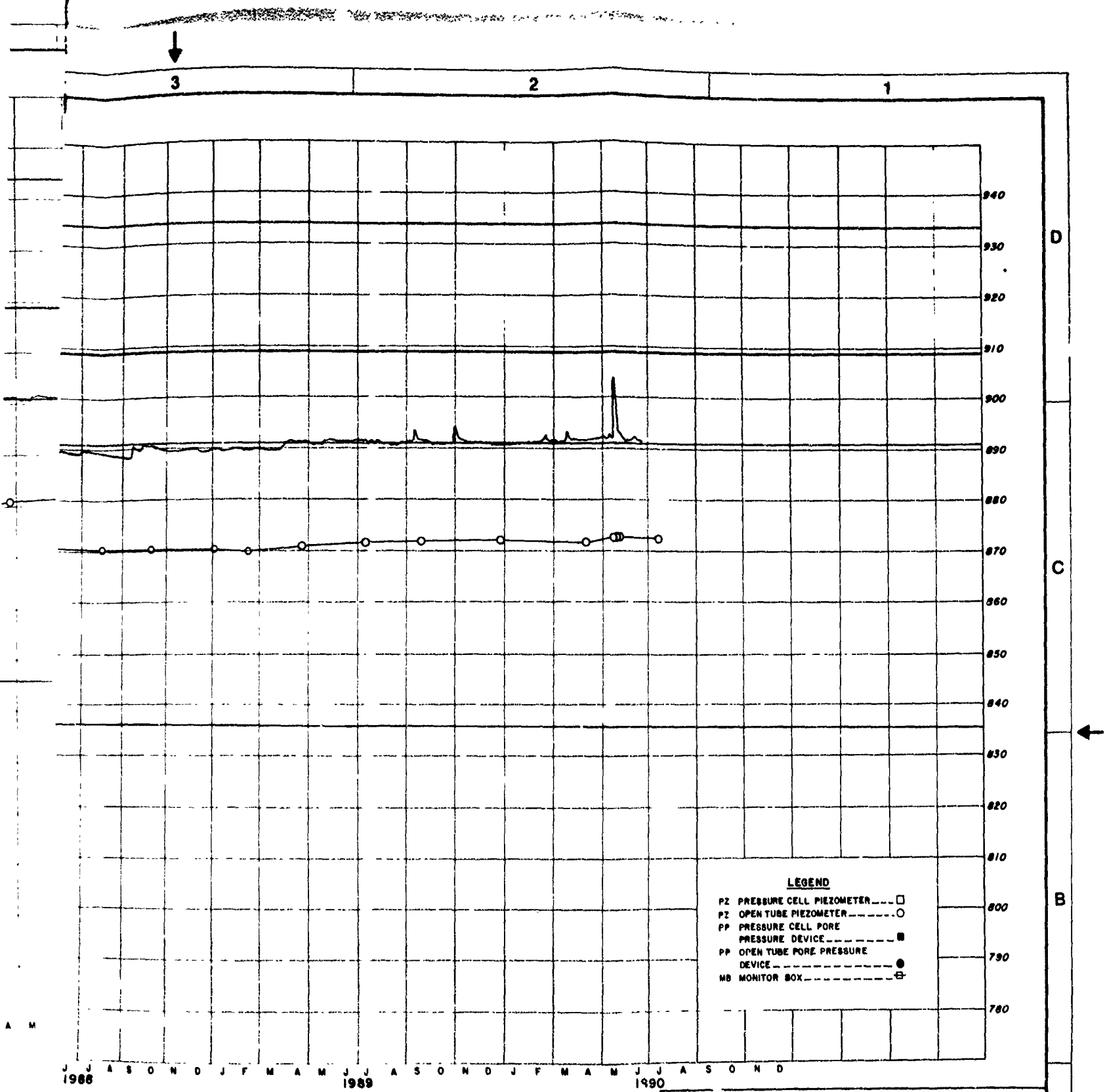
U S ARMY ENGINEER DISTRICT  
 CORPS OF ENGINEERS  
 KANSAS CITY, MISSOURI

Designed by: RGD	<b>LITTLE BLUE RIVER, MISSOURI          LONGVIEW LAKE          EMBANKMENT CRITERIA AND          PERFORMANCE REPORT          ALIGNMENT MONUMENTS          HORIZONTAL AND VERTICAL DATA PLOTS          MONUMENTS B-1, B-2 AND B-3</b>	Scale AS SHOWN	Sheet number
Drawn by: EDP		Date DECEMBER 1986	
Checked by: RLA			
Submitted by: WBE			

File No. **RBL-3-1576**







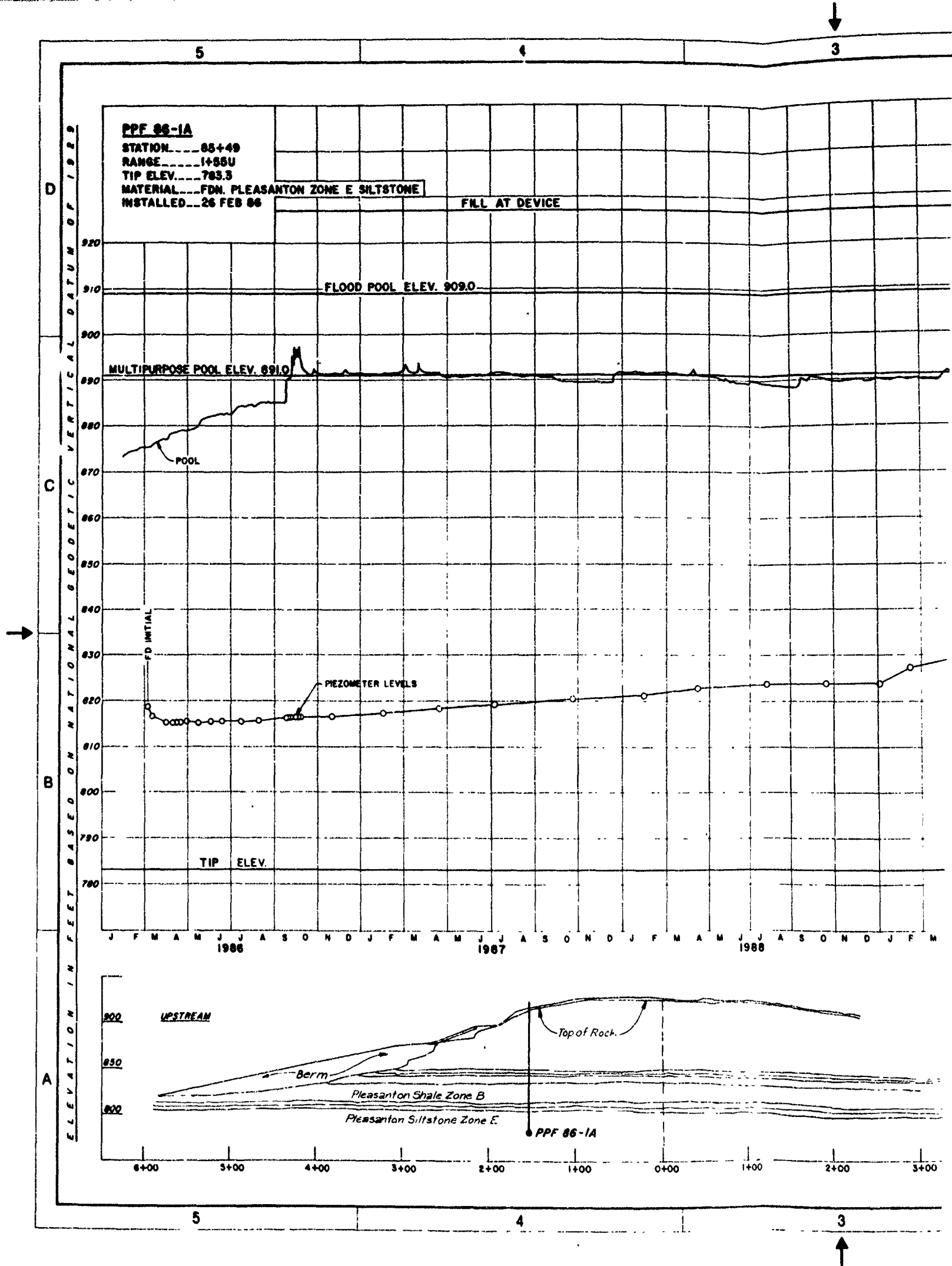
Symbol	Revisions	Date	Approved
	Revised March 1990		

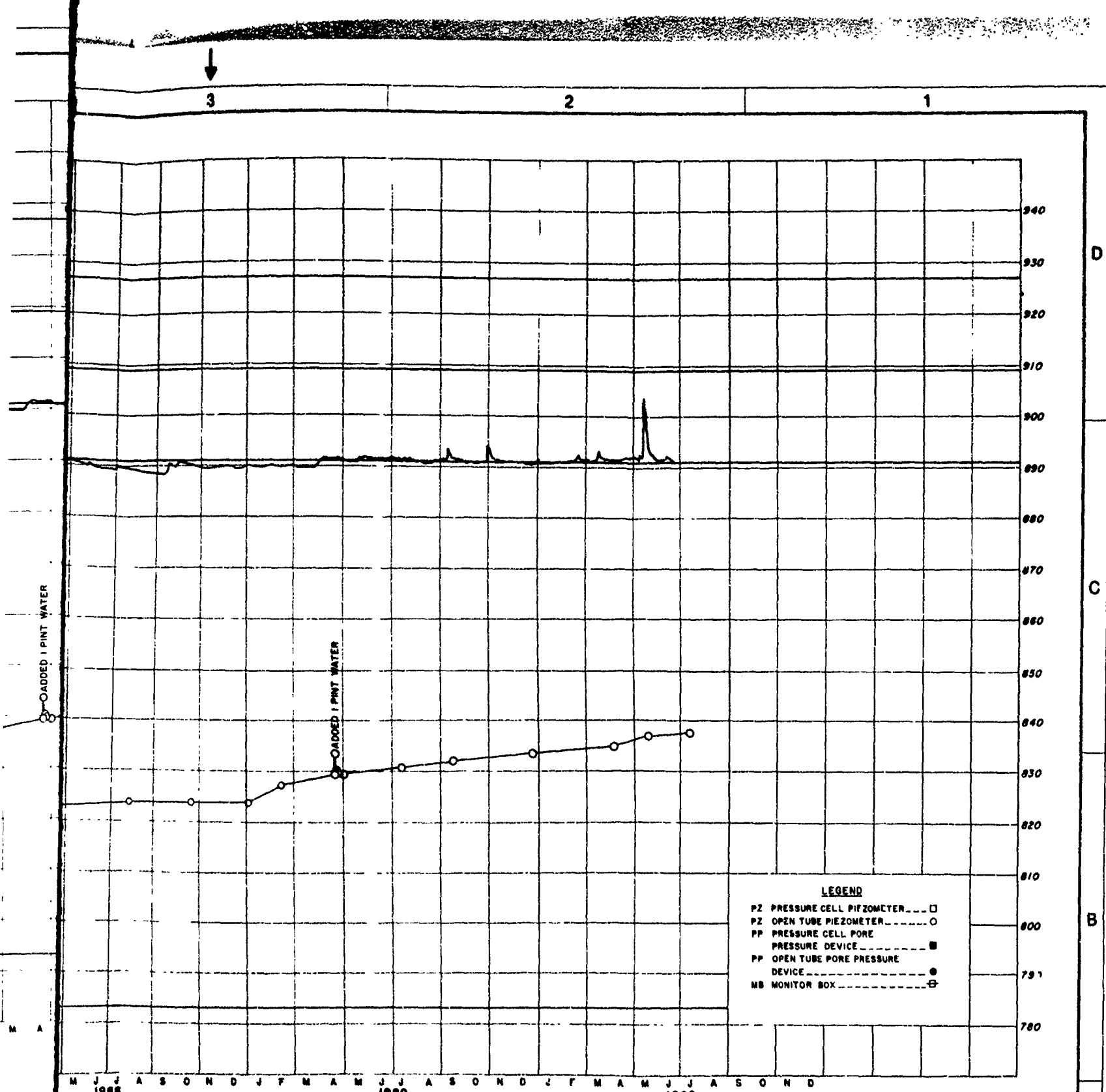
U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: RGD  
DRAWN BY: PST  
CHECKED BY: RLA  
SUBMITTED BY: WBE

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
PIEZOMETER PLOTS  
OPEN TUBE PZ  
PPF 84-1

Scale: AS SHOWN  
Date: DECEMBER 1986  
Sheet number:   
File No: RBL-3-1577



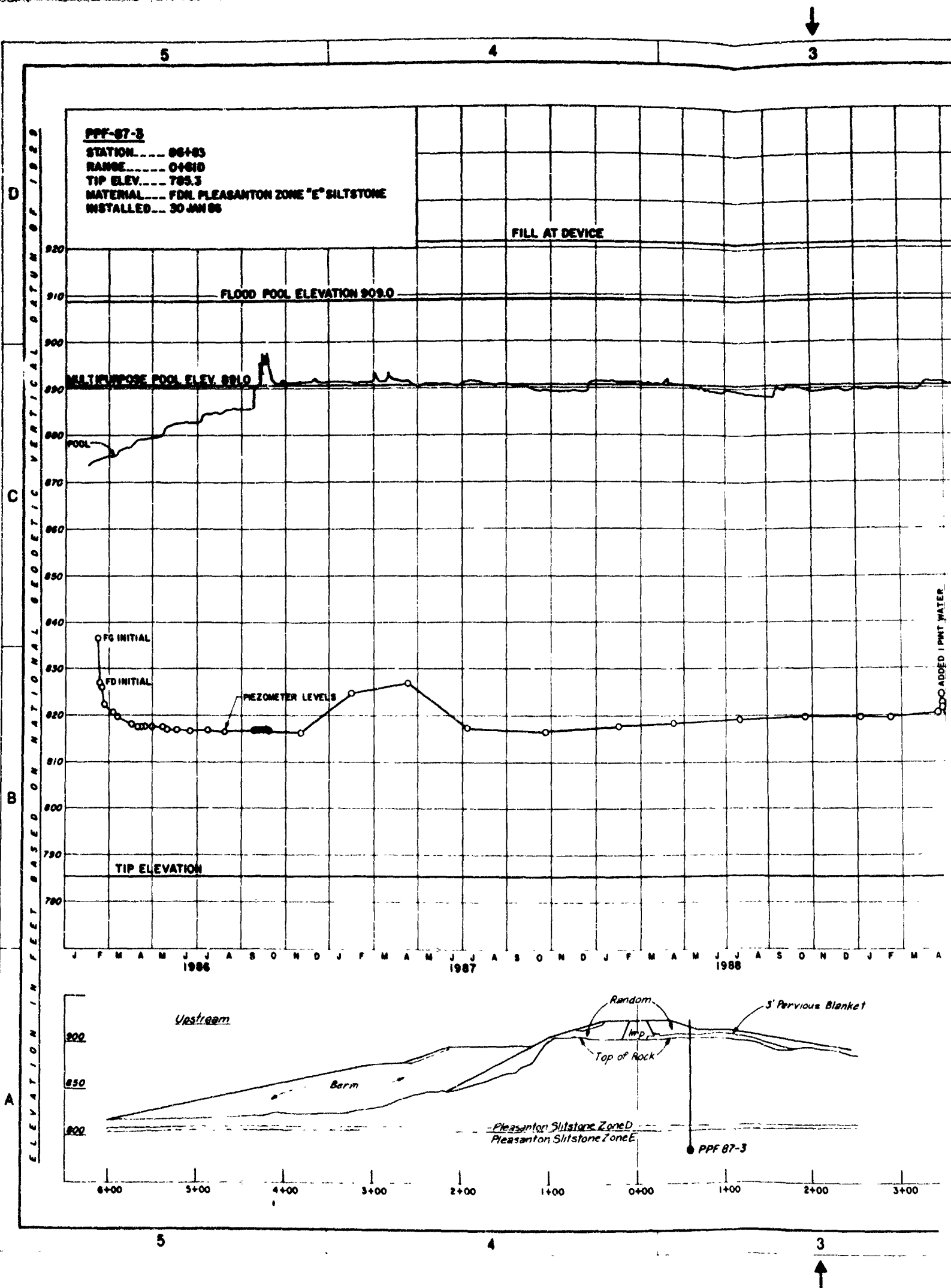


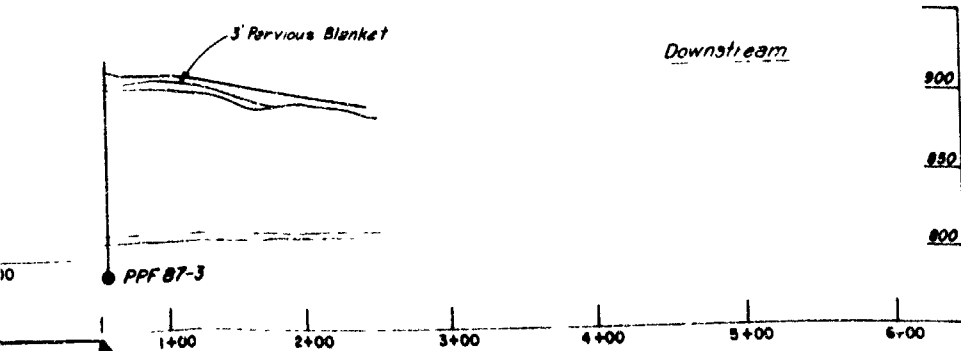
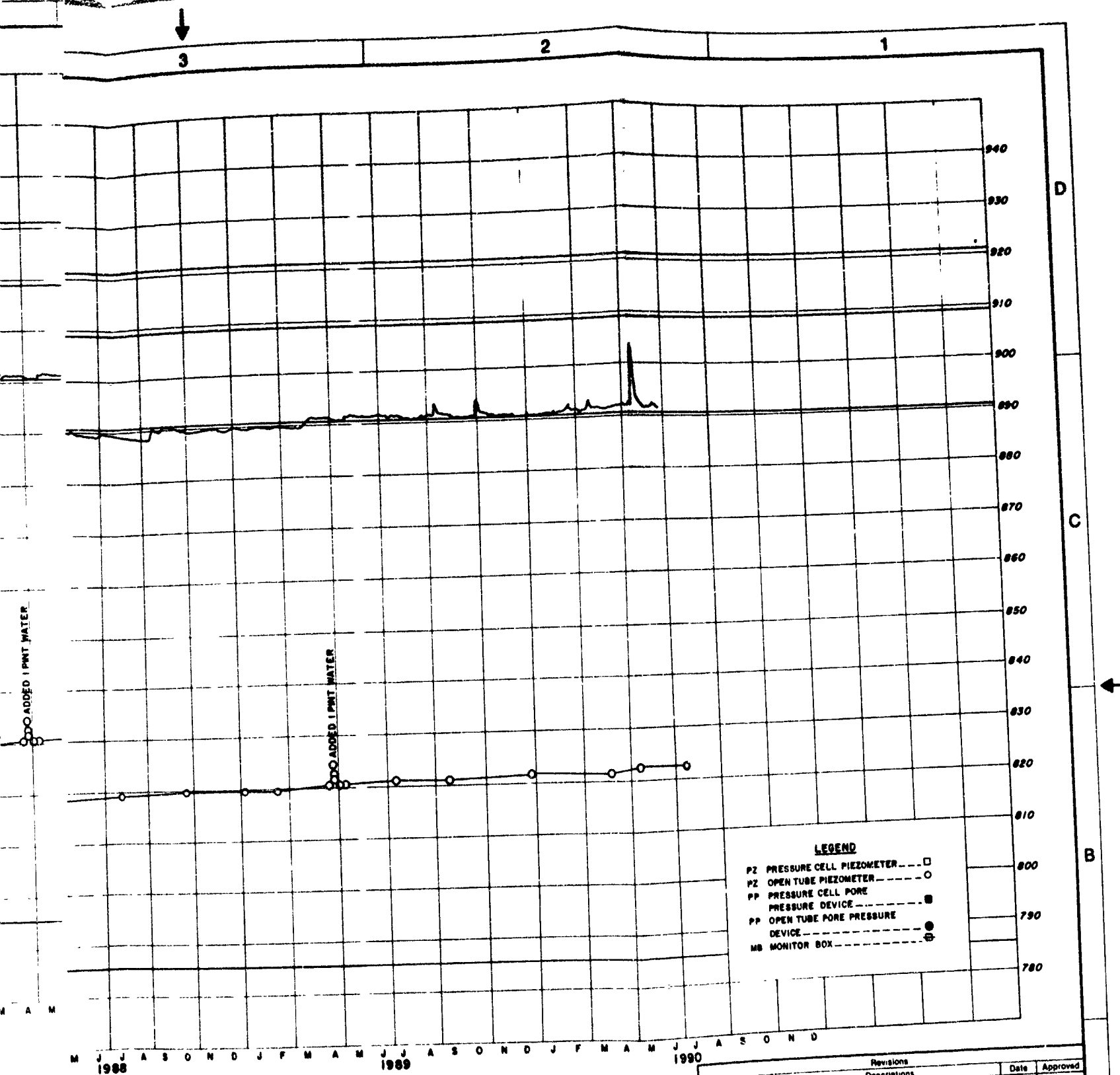
LEGEND	
PZ	PRESSURE CELL PIEZOMETER
PZ	OPEN TUBE PIEZOMETER
PP	PRESSURE CELL PORE PRESSURE DEVICE
PP	OPEN TUBE PORE PRESSURE DEVICE
MB	MONITOR BOX

Revisions			
Symbol	Descriptions	Date	Approved
	Revised March 1990		

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by H. D. G.	<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT PIEZOMETER PLOTS OPEN TUBE PZ PPF 86-1A</p>	Scale AS SHOWN	Sheet Number
Drawn by PST		Date DECEMBER 1988	
Checked by RLA		File No RBL-3-1578	
Submitted by WBE			





Symbol	Revisions	Date	Approved
	Revised March 1990		

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT

PIEZOMETER PLOTS  
OPEN TUBE PZ  
PPF 87-3

Designed by: NGD

Drawn by: P S T

Checked by: R L A

Submitted by: W B E

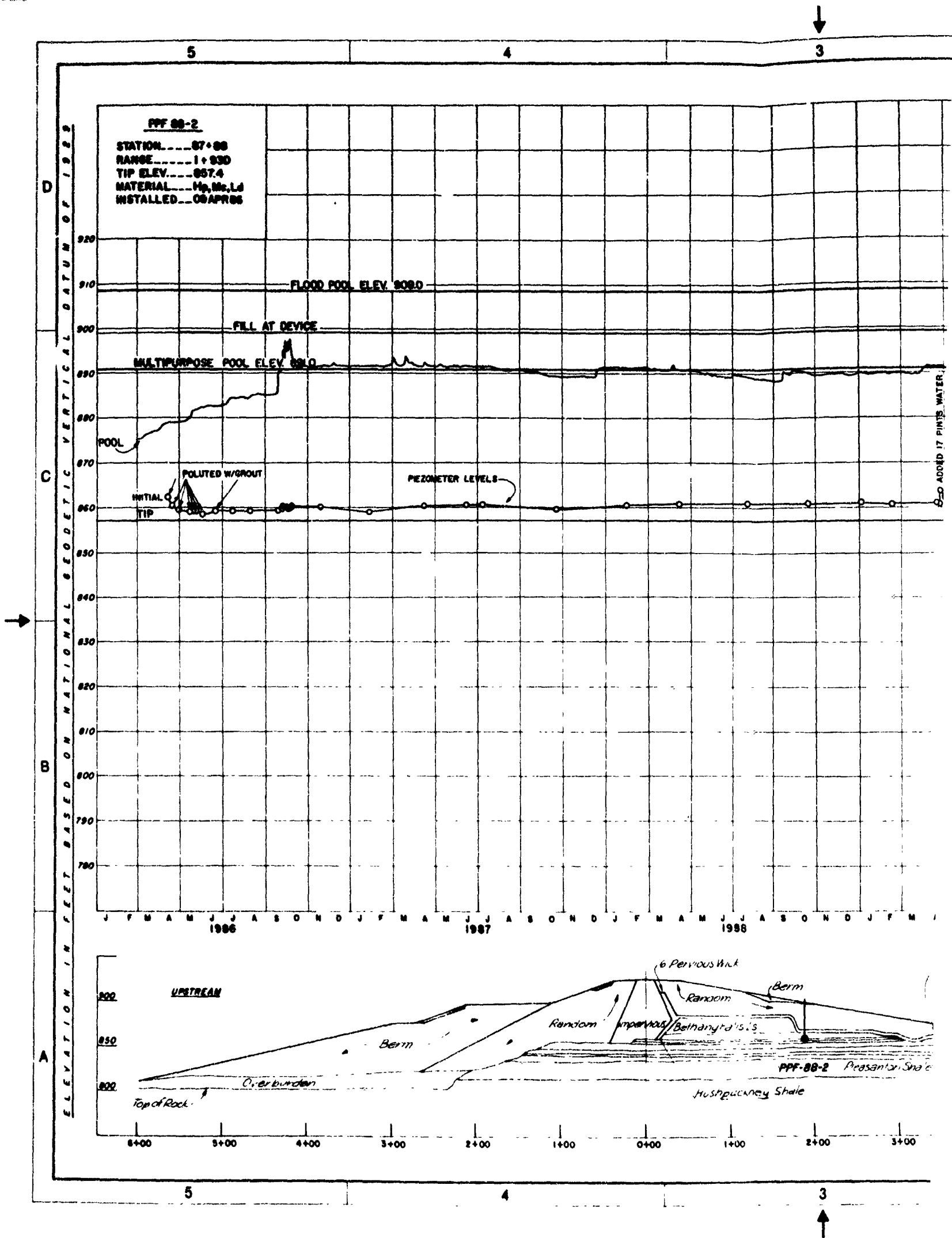
Scale: AS SHOWN

Date: DECEMBER 1986

Sheet number:

File No: RBL-3-1579

PLATE NO. 103



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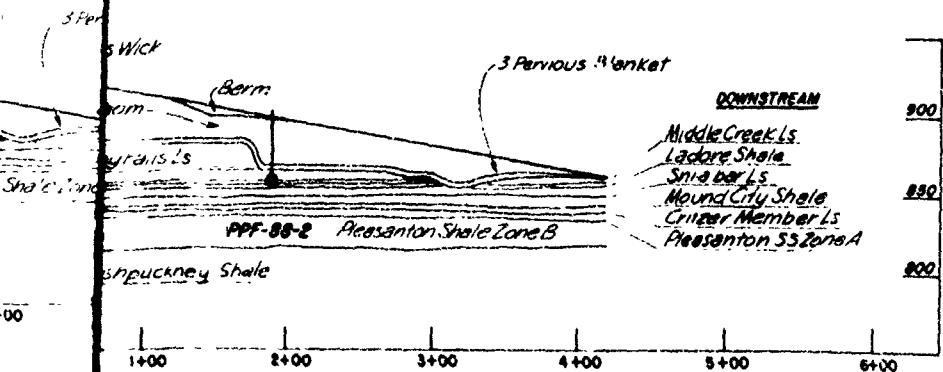
ADDED 17 PINTS WATER

ADDED 17 PINTS WATER

LEGEND

- PZ PRESSURE CELL PIEZOMETER
- PZ OPENTUBE PIEZOMETER
- PP PRESSURE CELL PORE PRESSURE DEVICE
- PP OPENTUBE PORE PRESSURE DEVICE
- MB MONITOR BOX

1988 1989 1990



Symbol	Revisions	Descriptions	Date	Approved
	Revised	March 1990		

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: RGD  
DRAWN BY: FDN  
CHECKED BY: RLA  
SUBMITTED BY: WSE

LITTLE BLUE RIVER, MISSOURI  
EMBRACEMENT CHANNEL AND  
PERFORMANCE REPORT  
PIEZOMETER PLOTS  
OPEN TUBE PZ  
PPF-88-2

Scale AS SHOWN  
Date DECEMBER 1988  
Drawn By  
Sheet number  
FILE NO. RBL-3-1580

PLATE NO. 104

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**PPF 88-3**  
 STATION: 88+02  
 RANGE: 8+20  
 TIP ELEV: 808.4  
 MATERIAL: PLS  
 INSTALLED: 25 APR 87

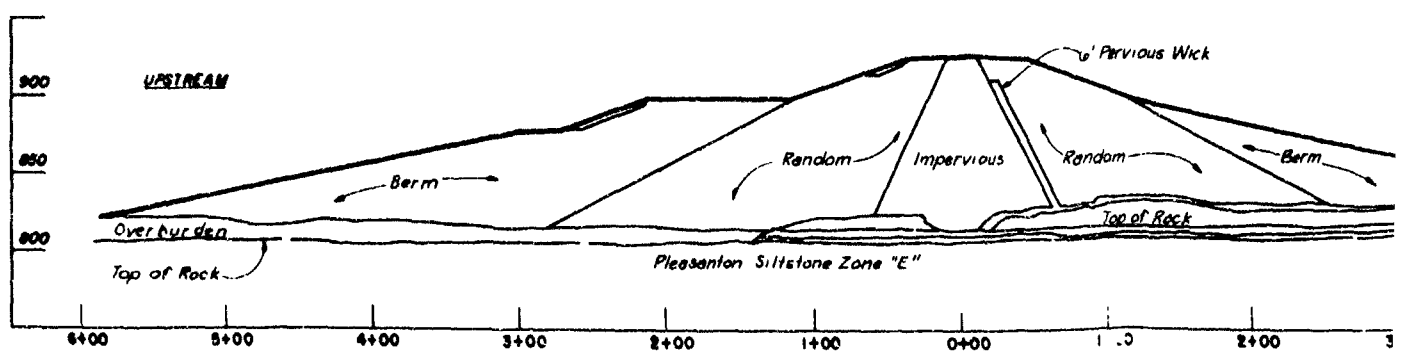
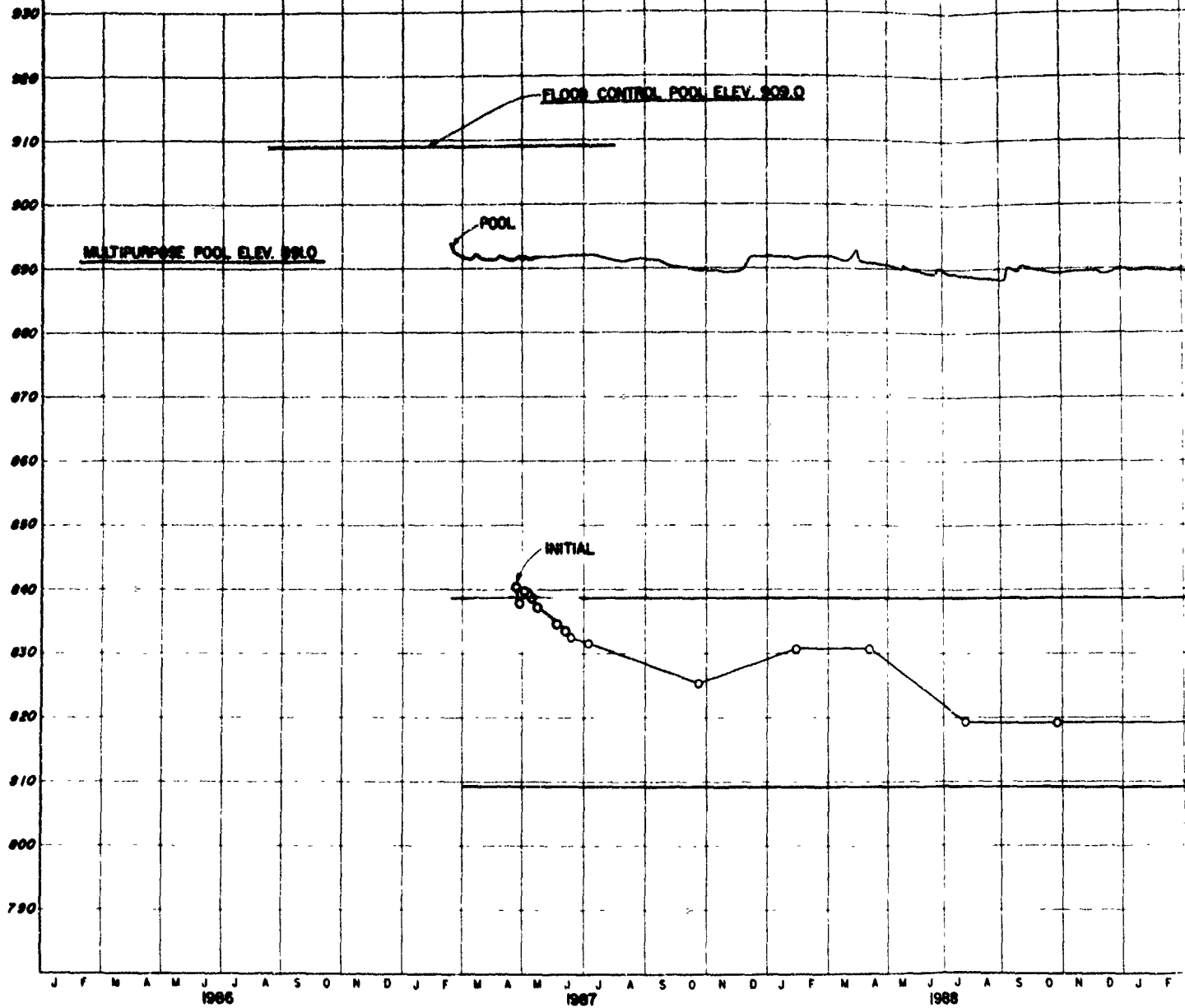
D

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ELEVATION IN FEET BASED ON NATIONAL GEODETIC VERTICAL DATUM OF 1988



5

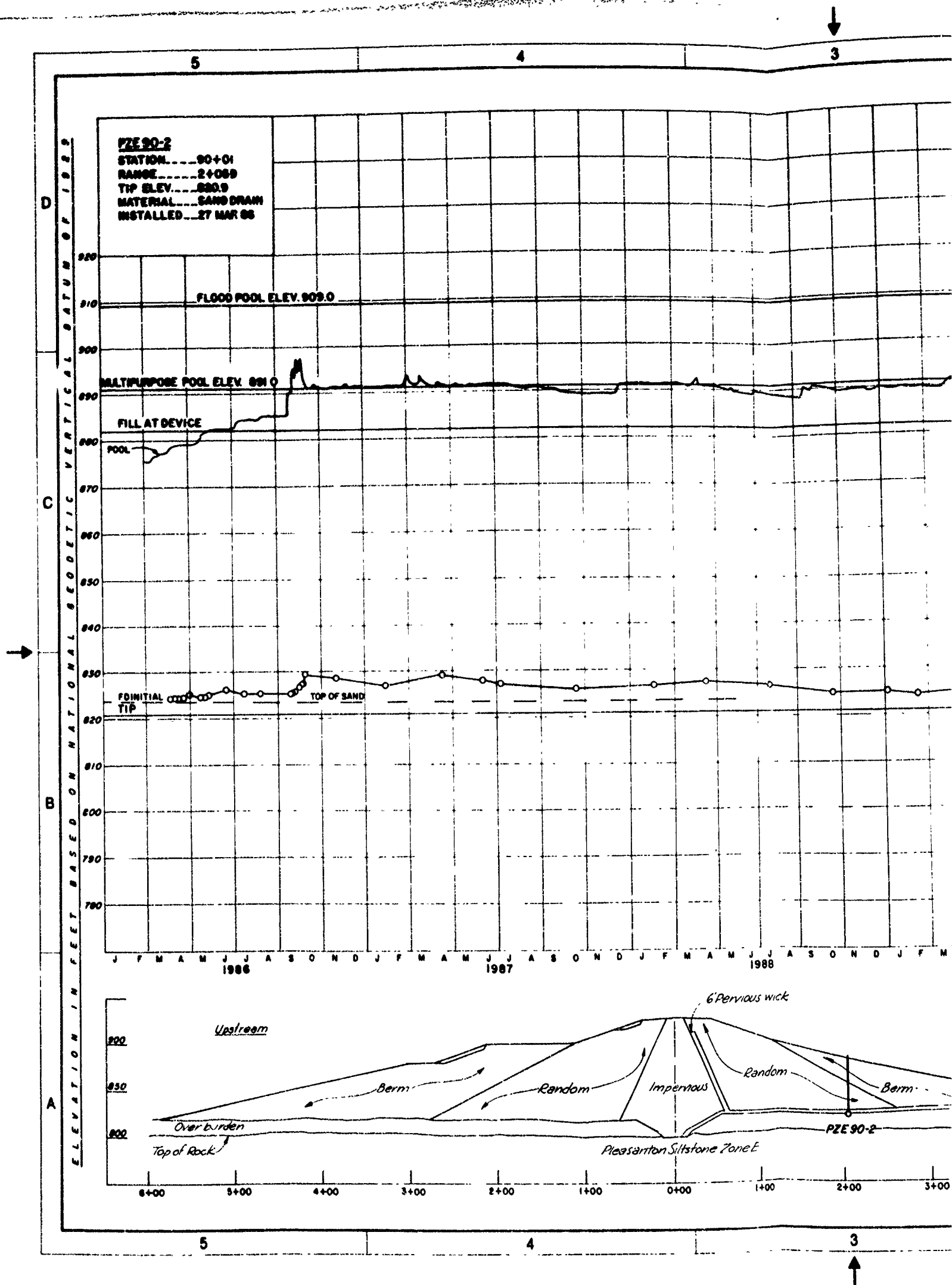
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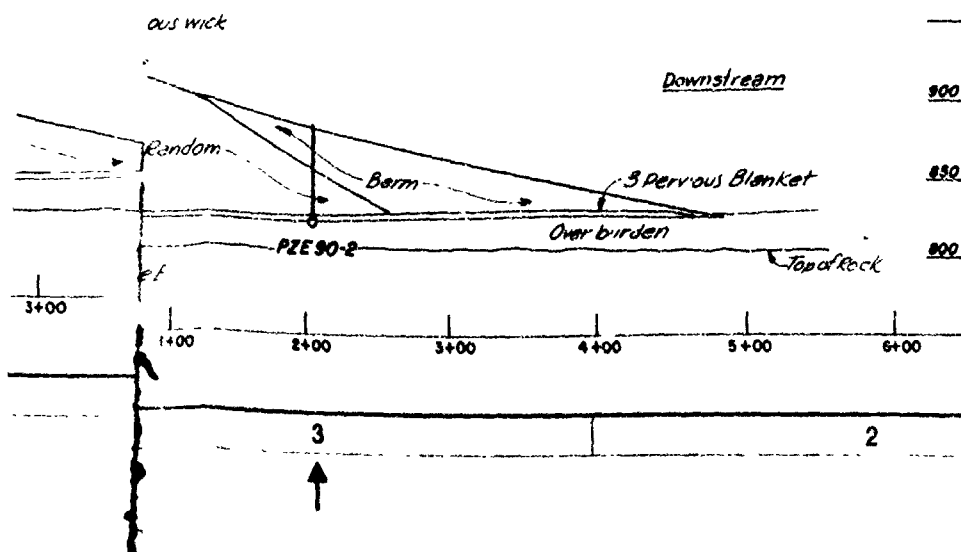
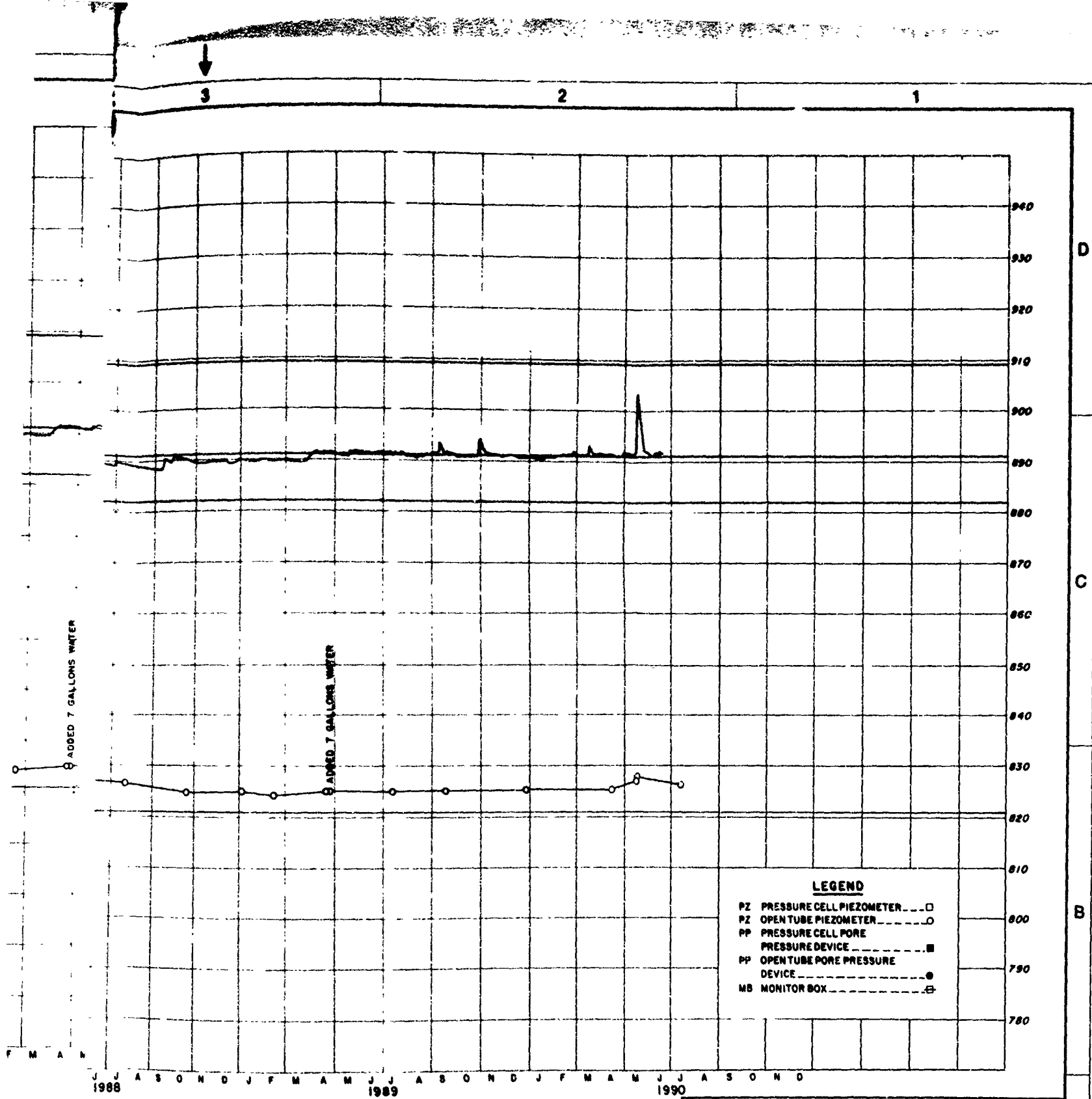
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Revisions			
Symbol	Descriptions	Date	Approved
	Revised March 1990		

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: RGD

Drawn by: PFL

Checked by: RLA

Submitted by: WEE

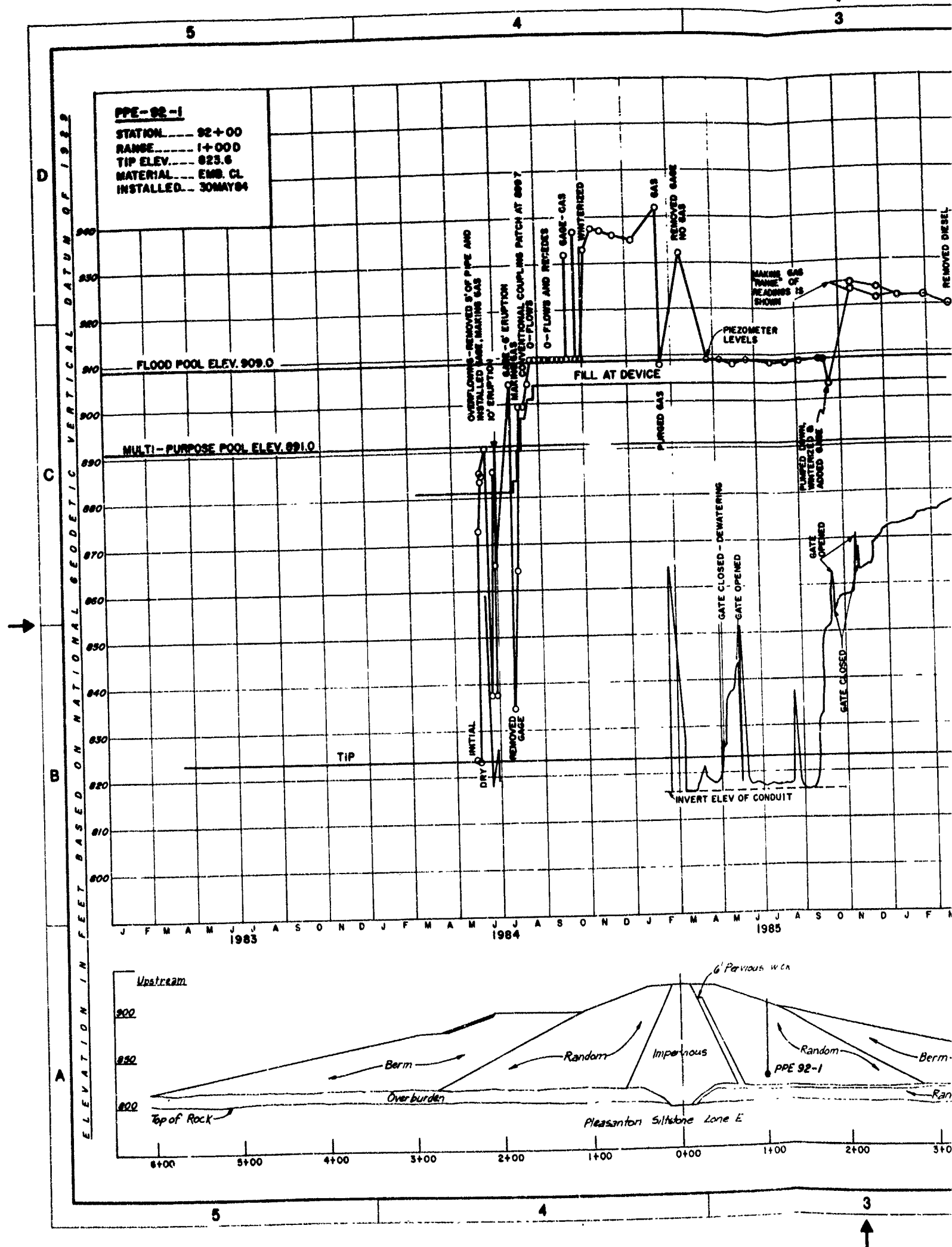
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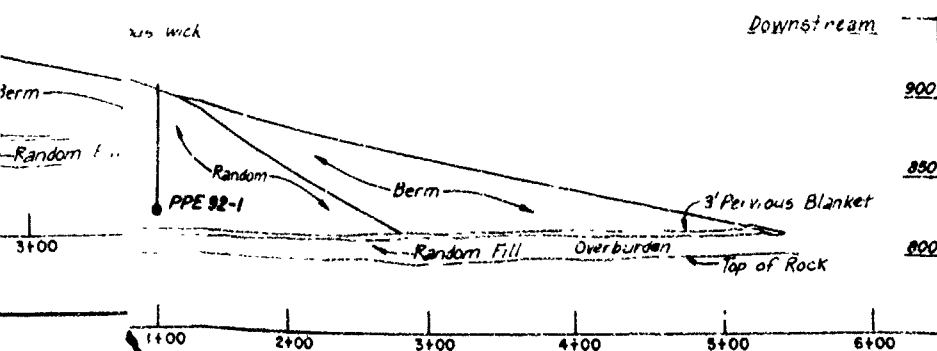
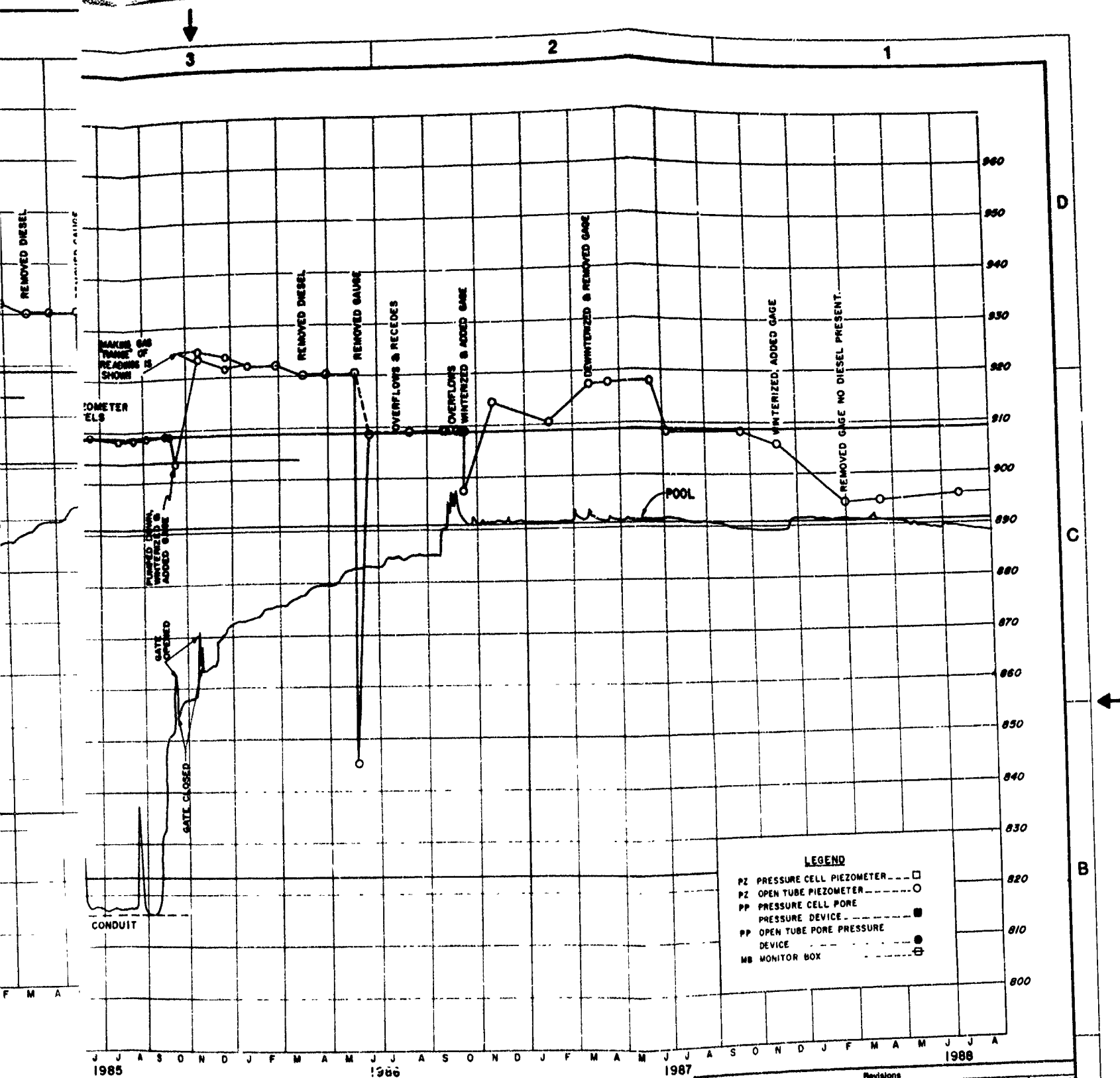
Date: DECEMBER 1988

Sheet number: 1

File No.: RBL-3-1582

PLATE NO. 106





Revisions		Date	Approved
Symbol	Descriptions		

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

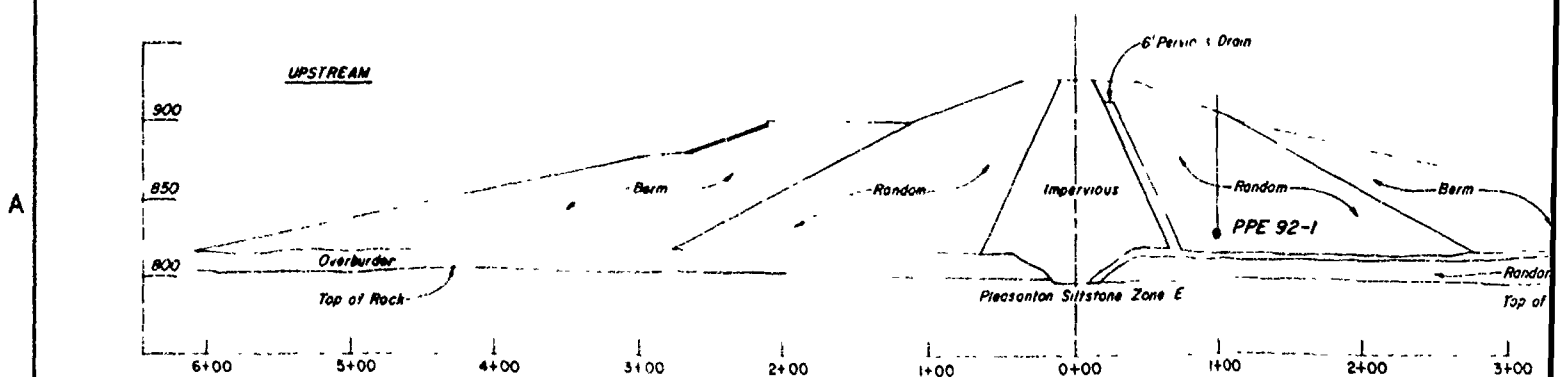
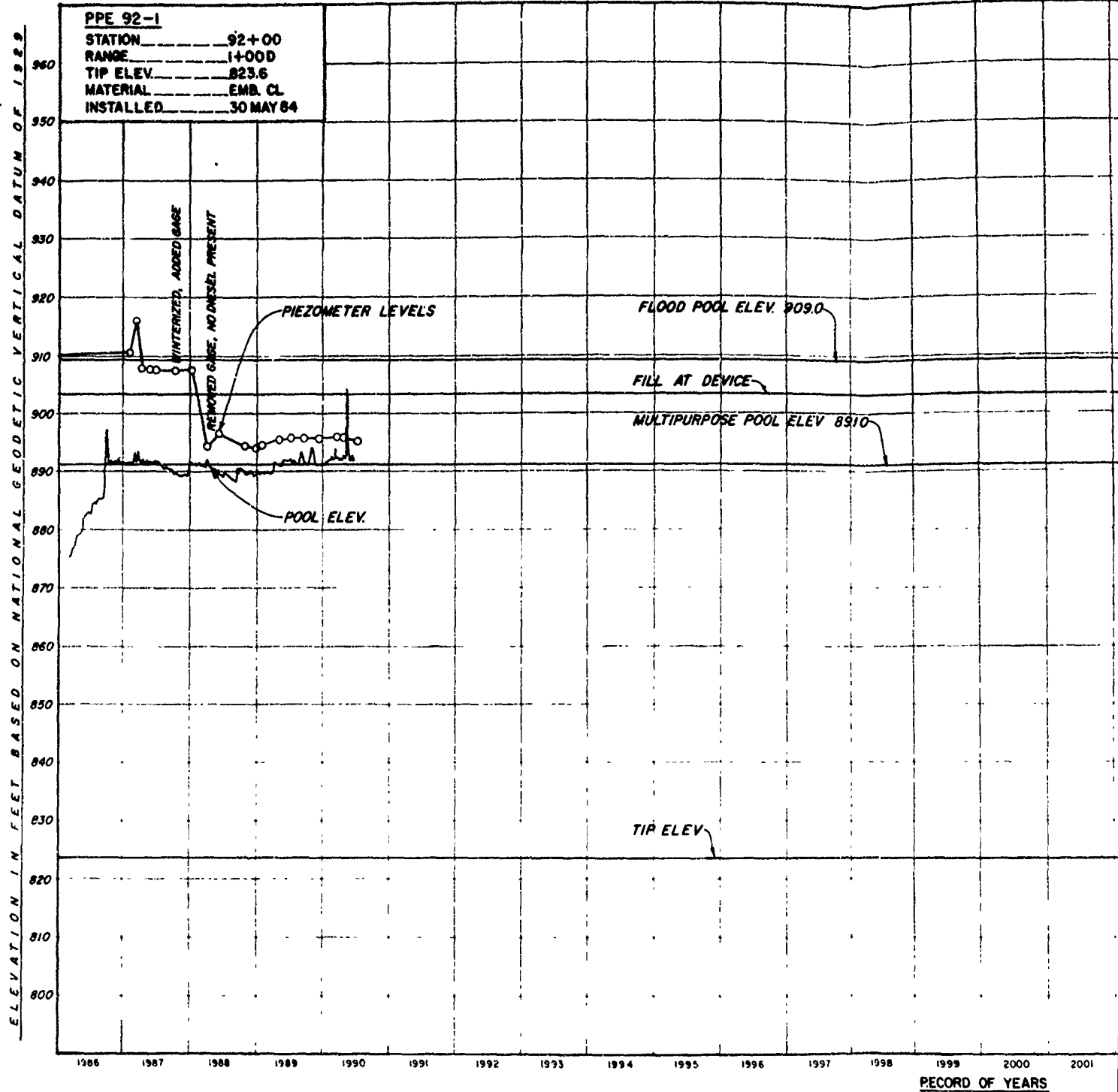
DESIGNED BY: RGD  
DRAWN BY: MJK  
CHECKED BY: RLA  
SUBMITTED BY: WBE

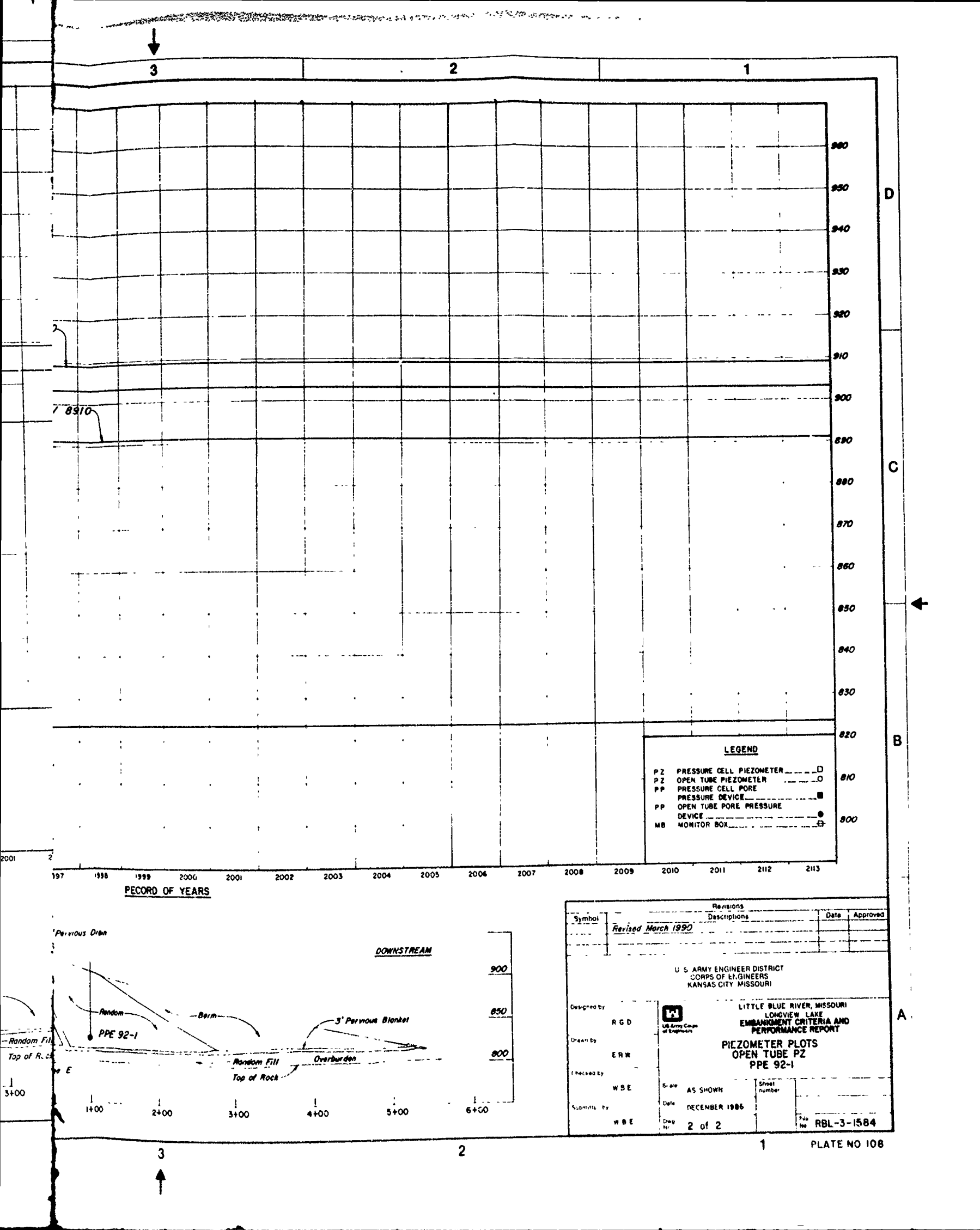
LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
PIEZOMETER PLOTS  
OPEN TUBE PZ  
PPE 92-1

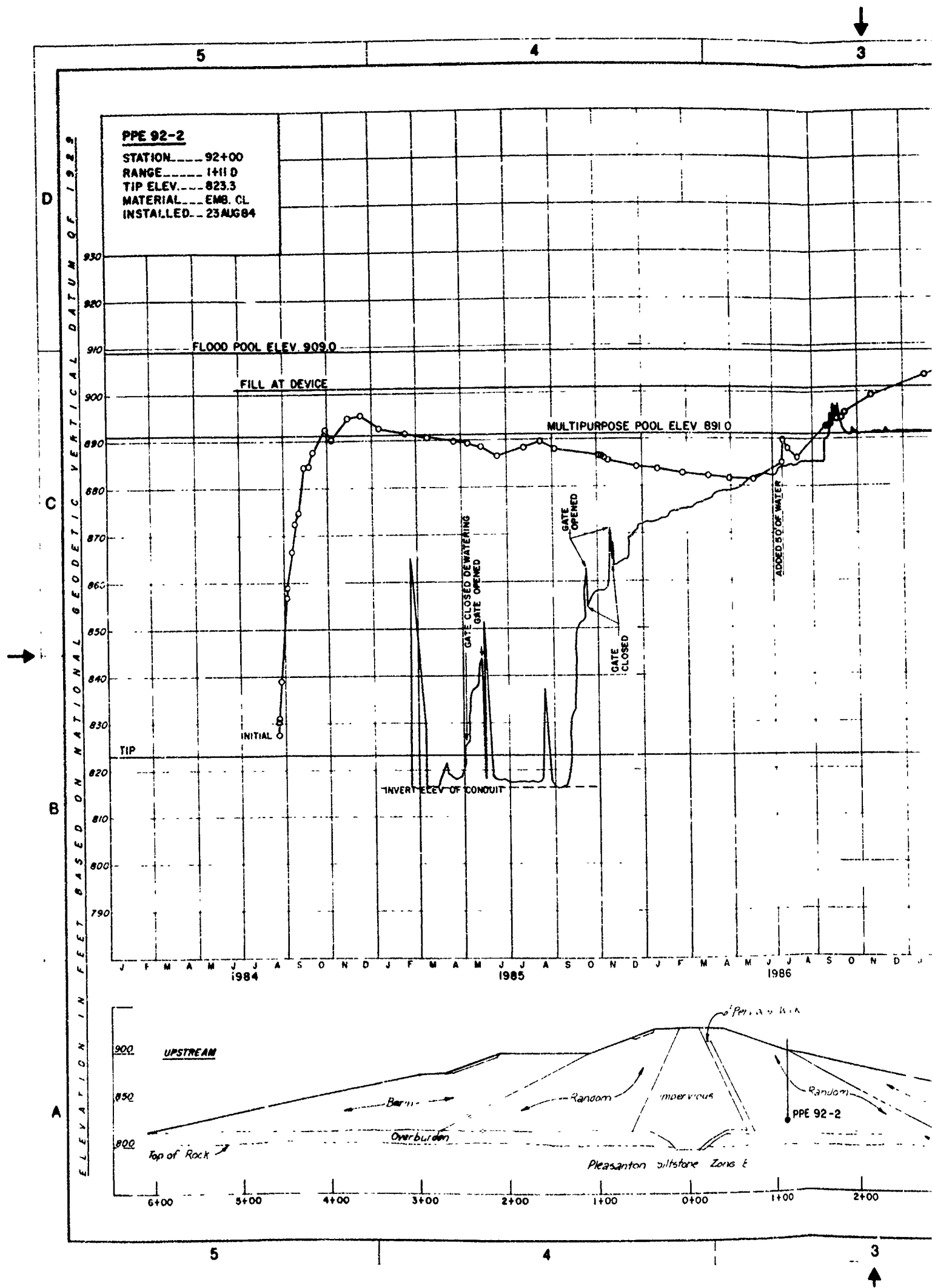
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Date: DECEMBER 1986  
Draw No: 1 OF 2

Sheet number: 1  
File No: RBL-3-1583

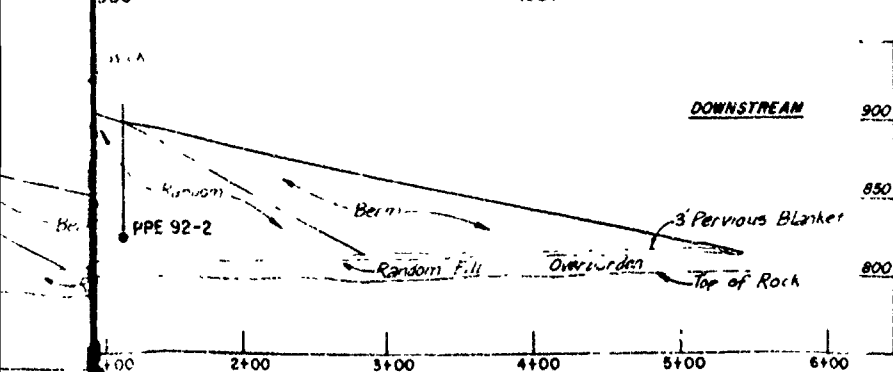
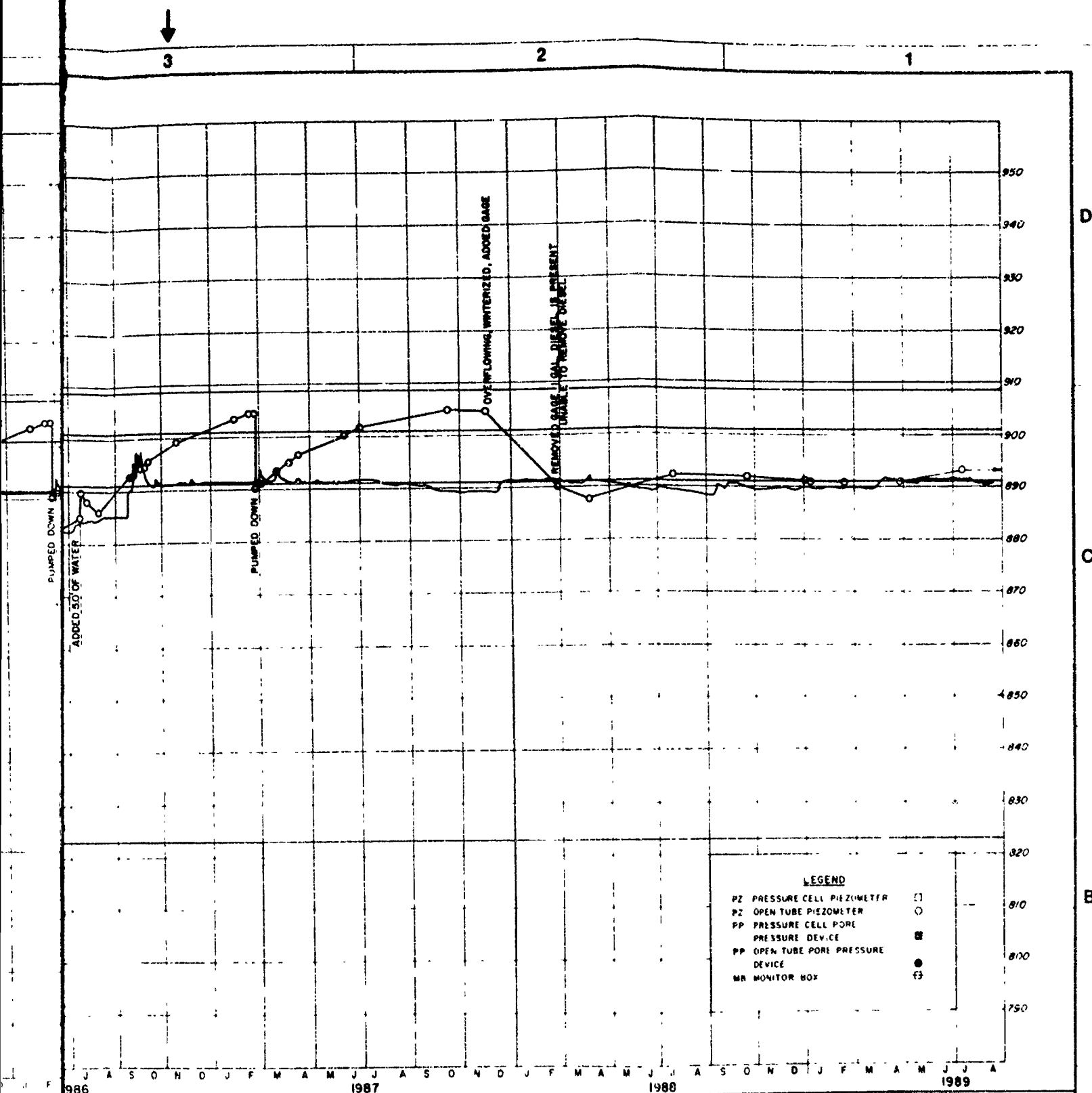
PLATE NO. 107











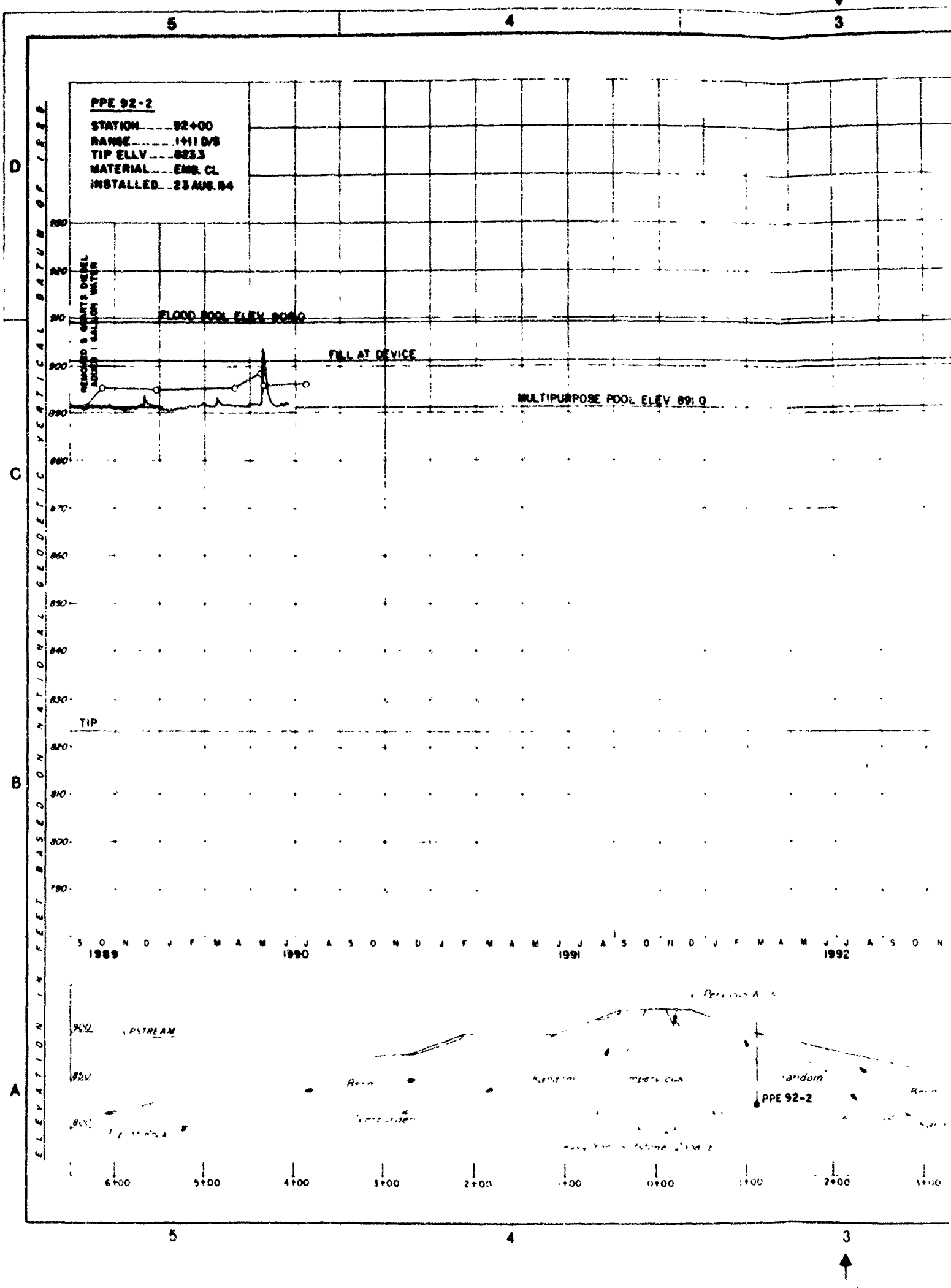
Revisions			
Symbol	Description	Date	Approved
	Revised March 1990		

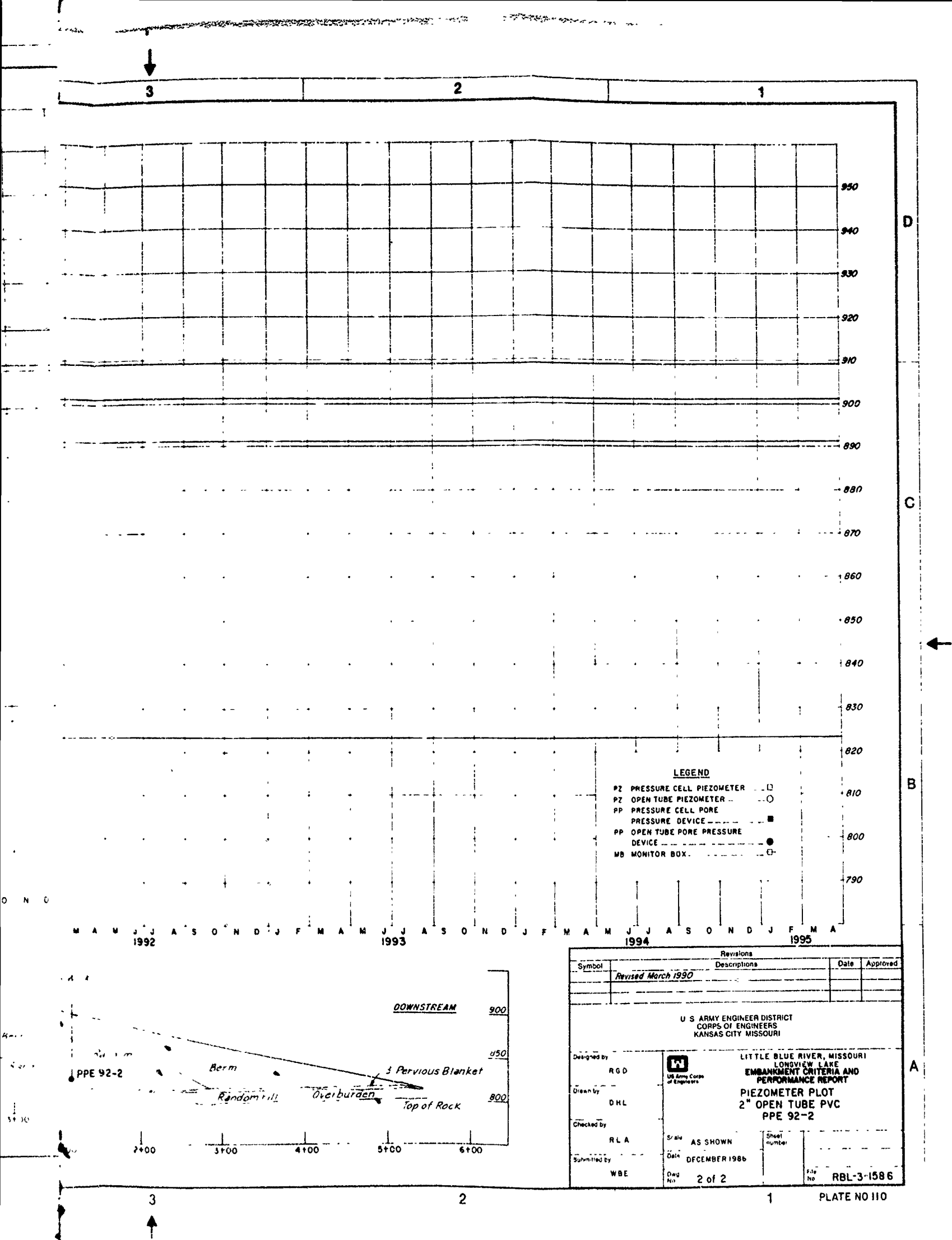
U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

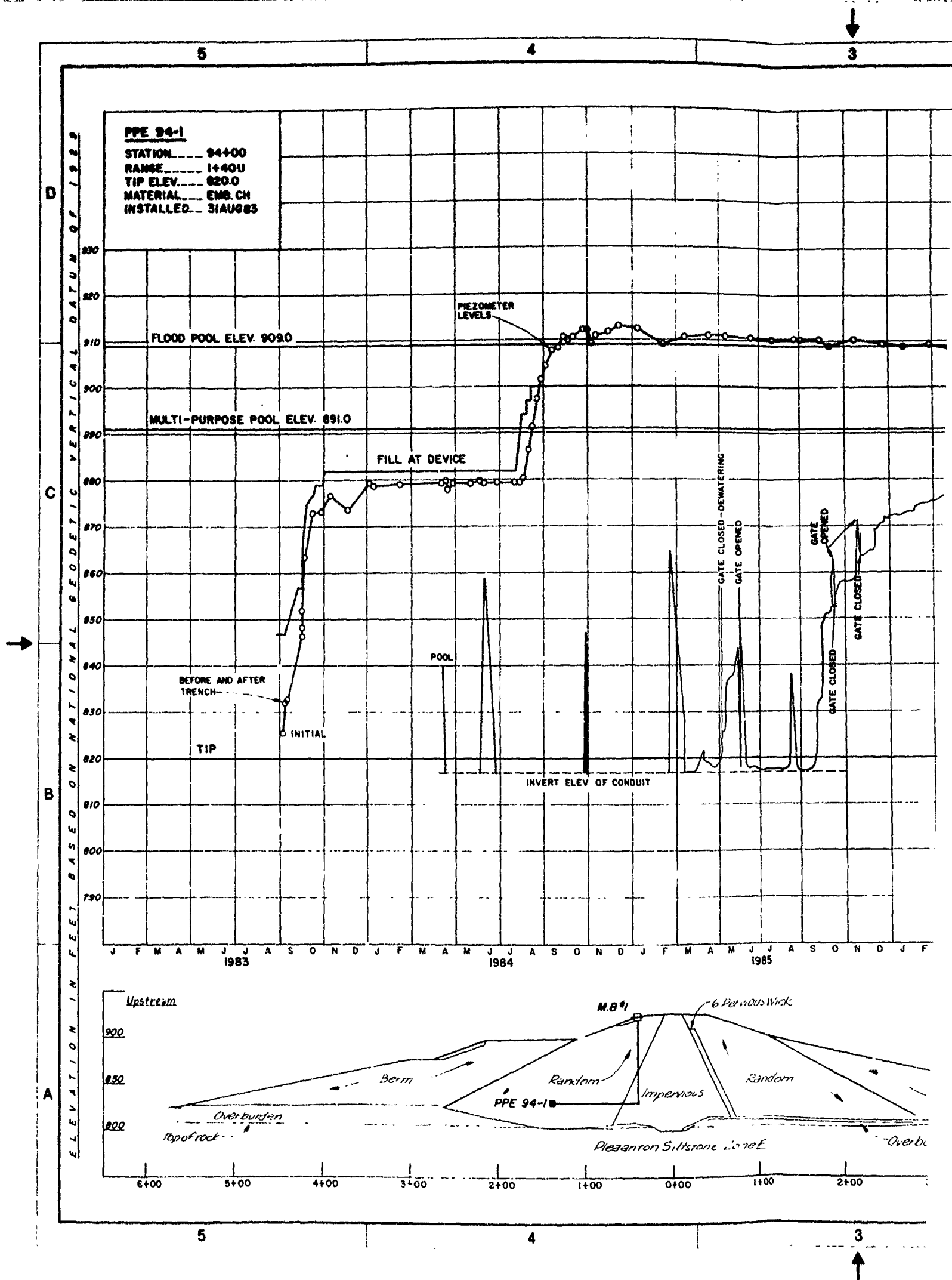
DESIGNED BY: RGD  
DRAWN BY: DML  
CHECKED BY: RLA  
SUBMITTED BY: WBE

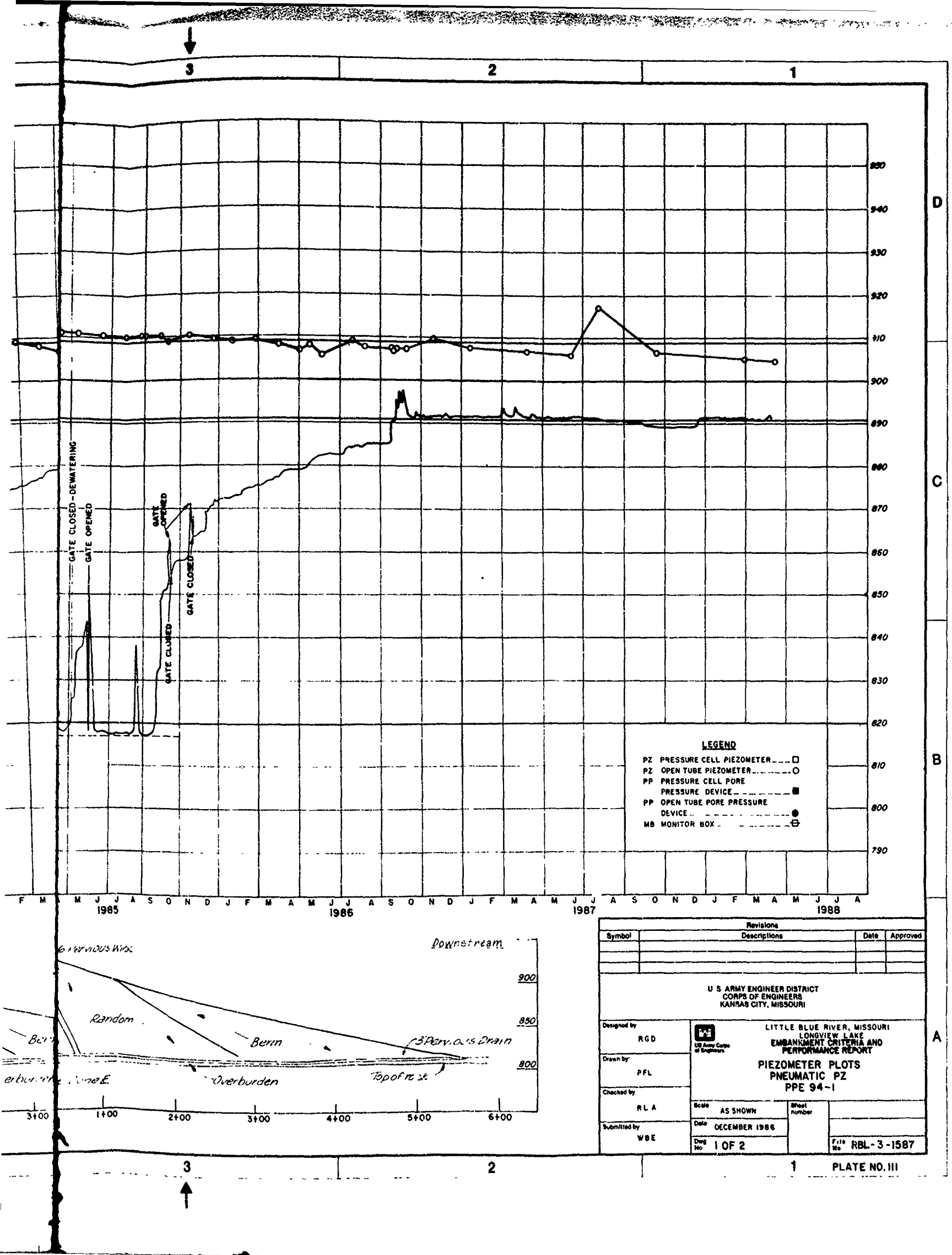
**LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
PIEZOMETER PLOT  
2" OPEN TUBE PVC  
PPE 92-2**

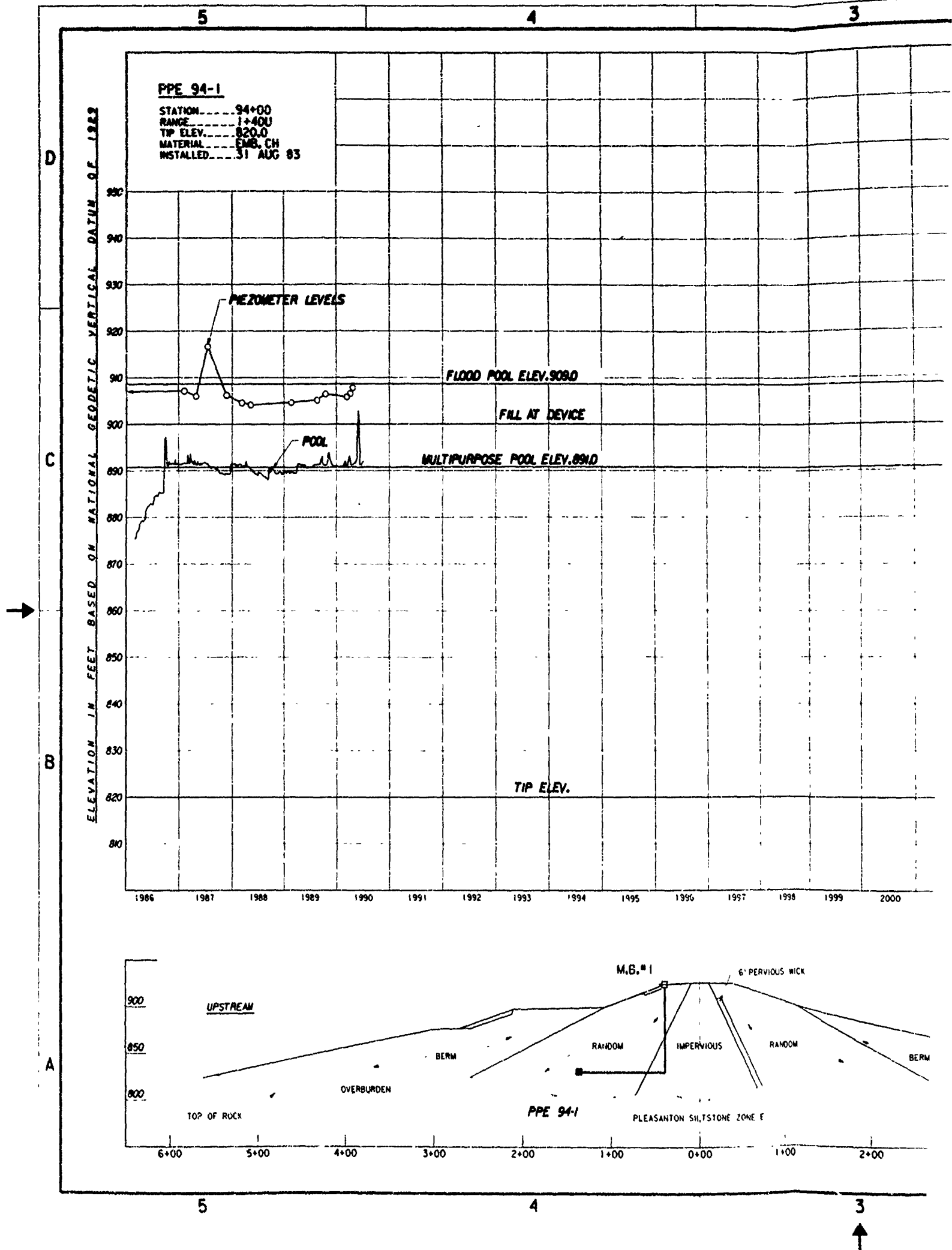
Scale: AS SHOWN  
Date: DECEMBER 1986  
Sheet: 1 of 2  
File No: RF-3-1585











VALUE E

VALUE ENGINEERING PAYS

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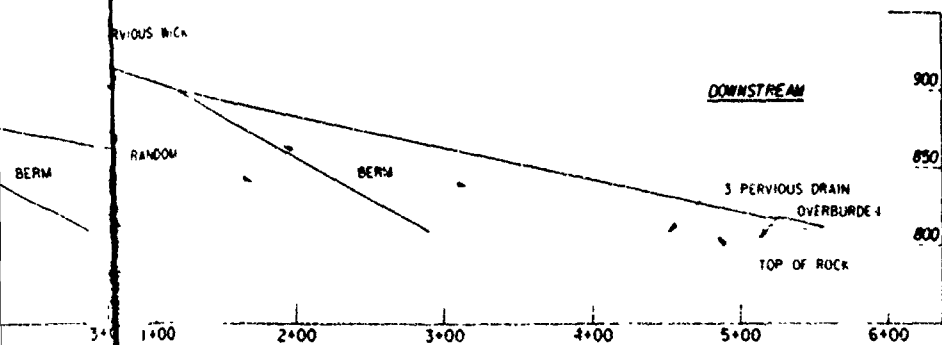
A

LEGEND

PZ PRESSURE CELL PIEZOMETER  
 PZ OPEN TUBE PIEZOMETER  
 PP PRESSURE CELL PORE  
 PP PRESSURE DEVICE  
 PP OPEN TUBE PORE PRESSURE  
 DEVICES  
 MB MONITOR BOX

□  
 ○  
 ■  
 ●  
 □

2001 1998 1999 2000 2001 2002 2003 2004 2005 2006 2007 2008 2009 2010 2011 2012 2013



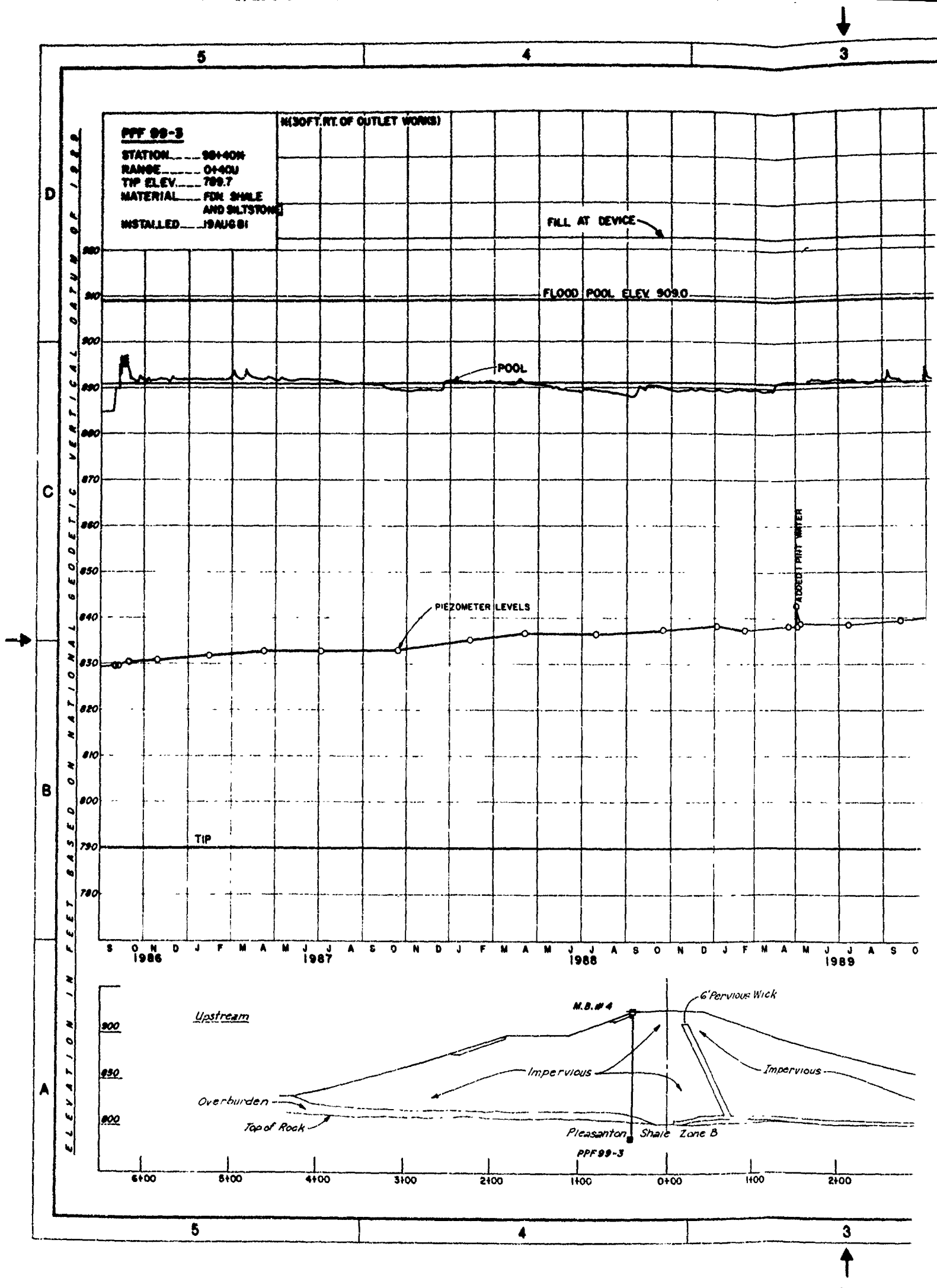
Symbol	Revisions	Date	Approved
	Descriptions		
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
Designed by	R.G.D.	LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT	
Drawn by	J.W.P.	PIEZOMETER PLOTS PNEUMATIC PZ PPE 94-1	
Checked by	W.B.E.	AS SHOWN	1211
Submitted by	W.B.F.	JULY 1990	PIEZOMETER PLOTS
		OF	RBL-3-1580

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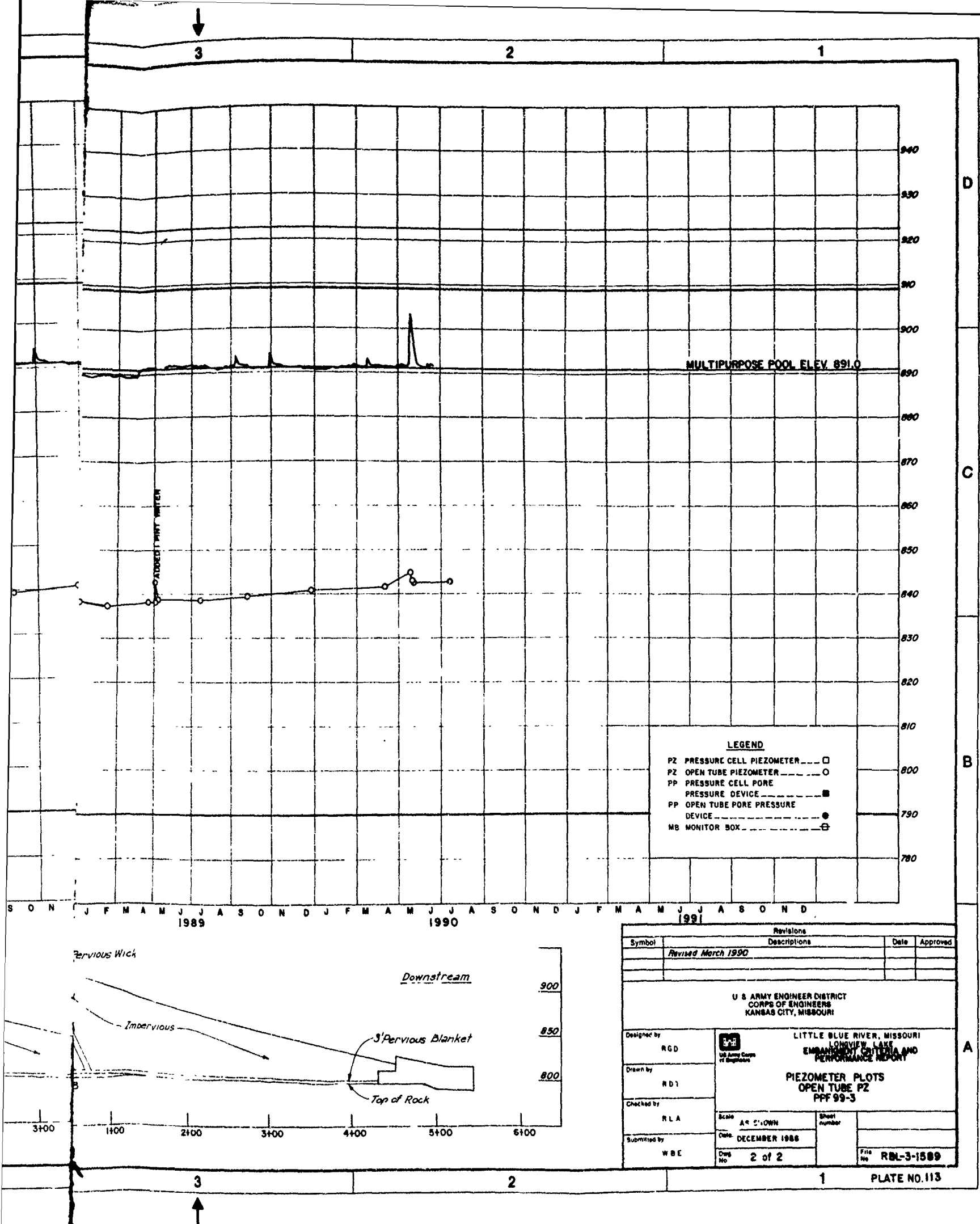
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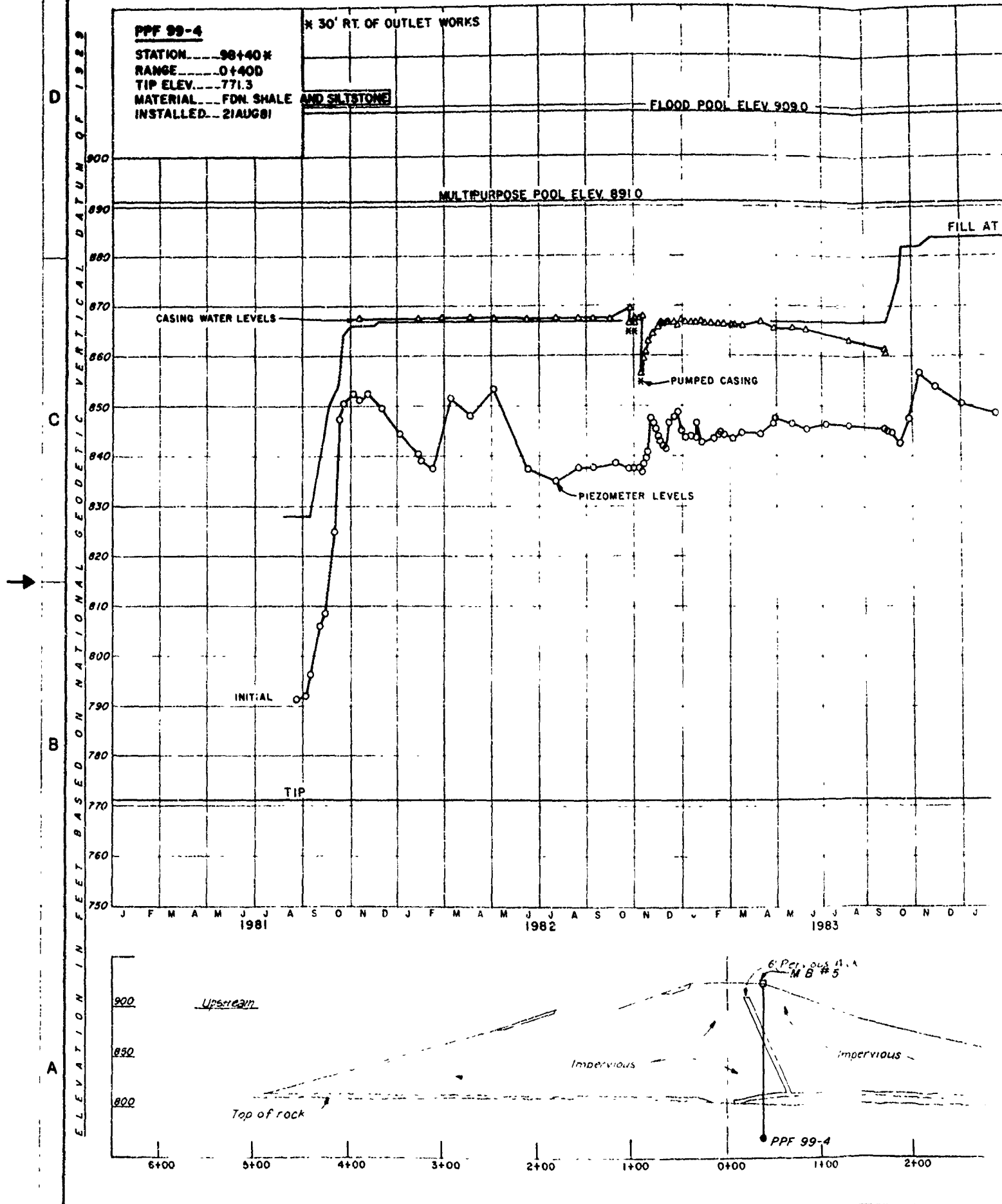
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PLATE NO. 112

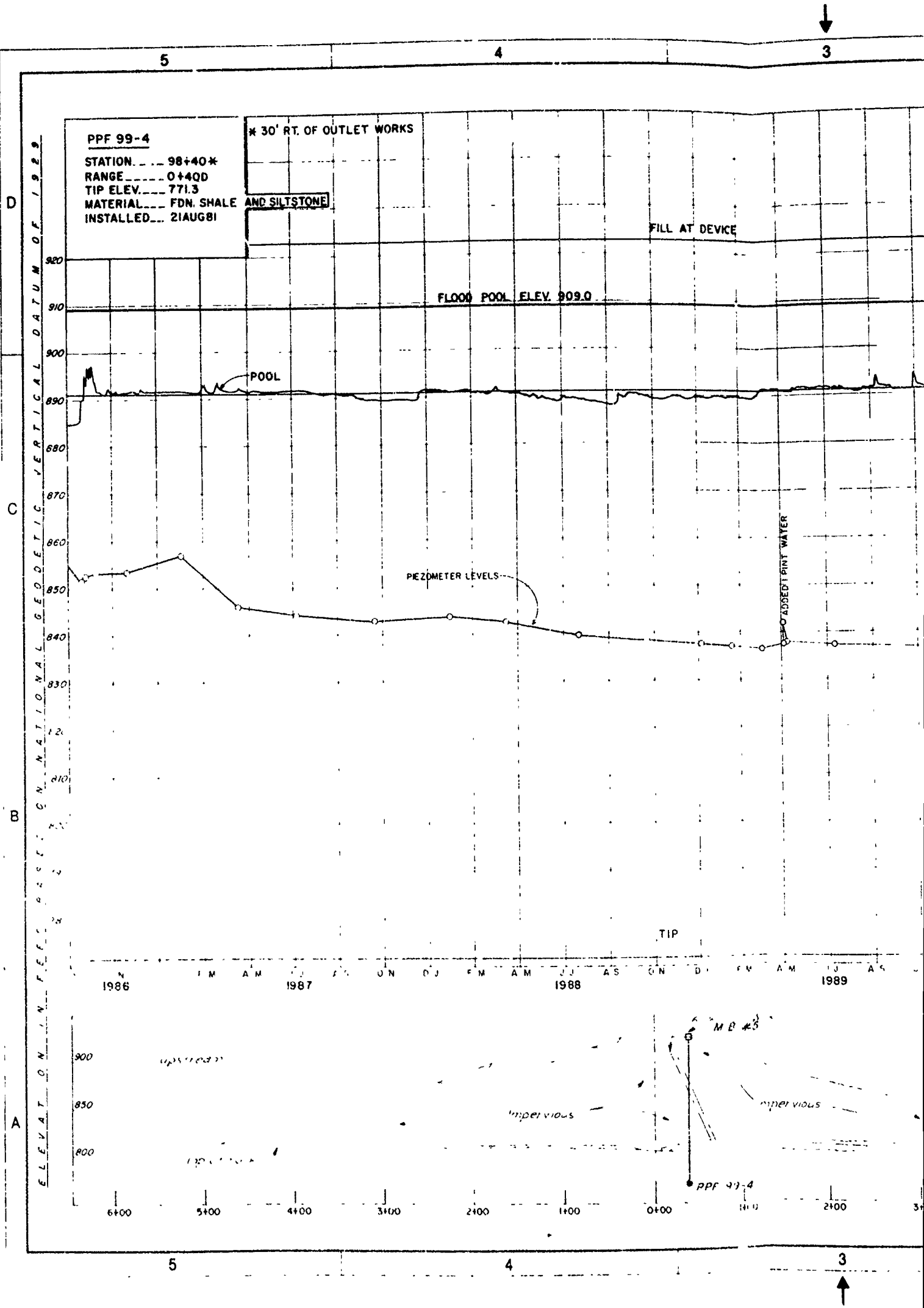


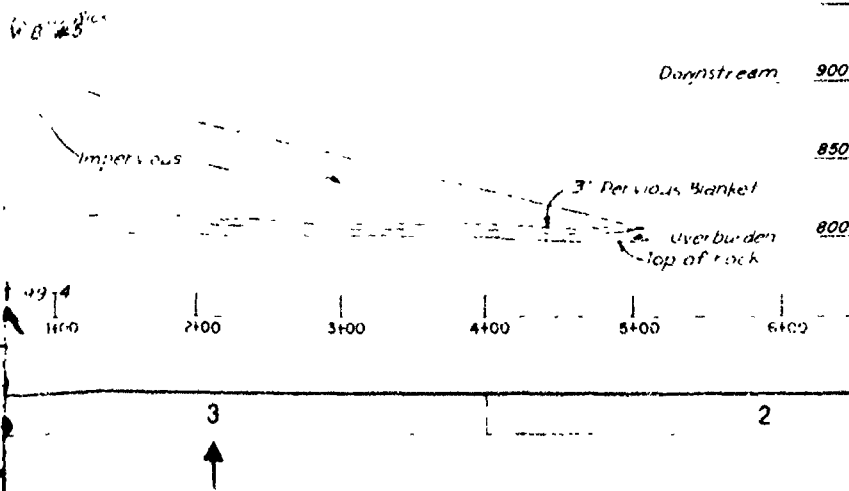
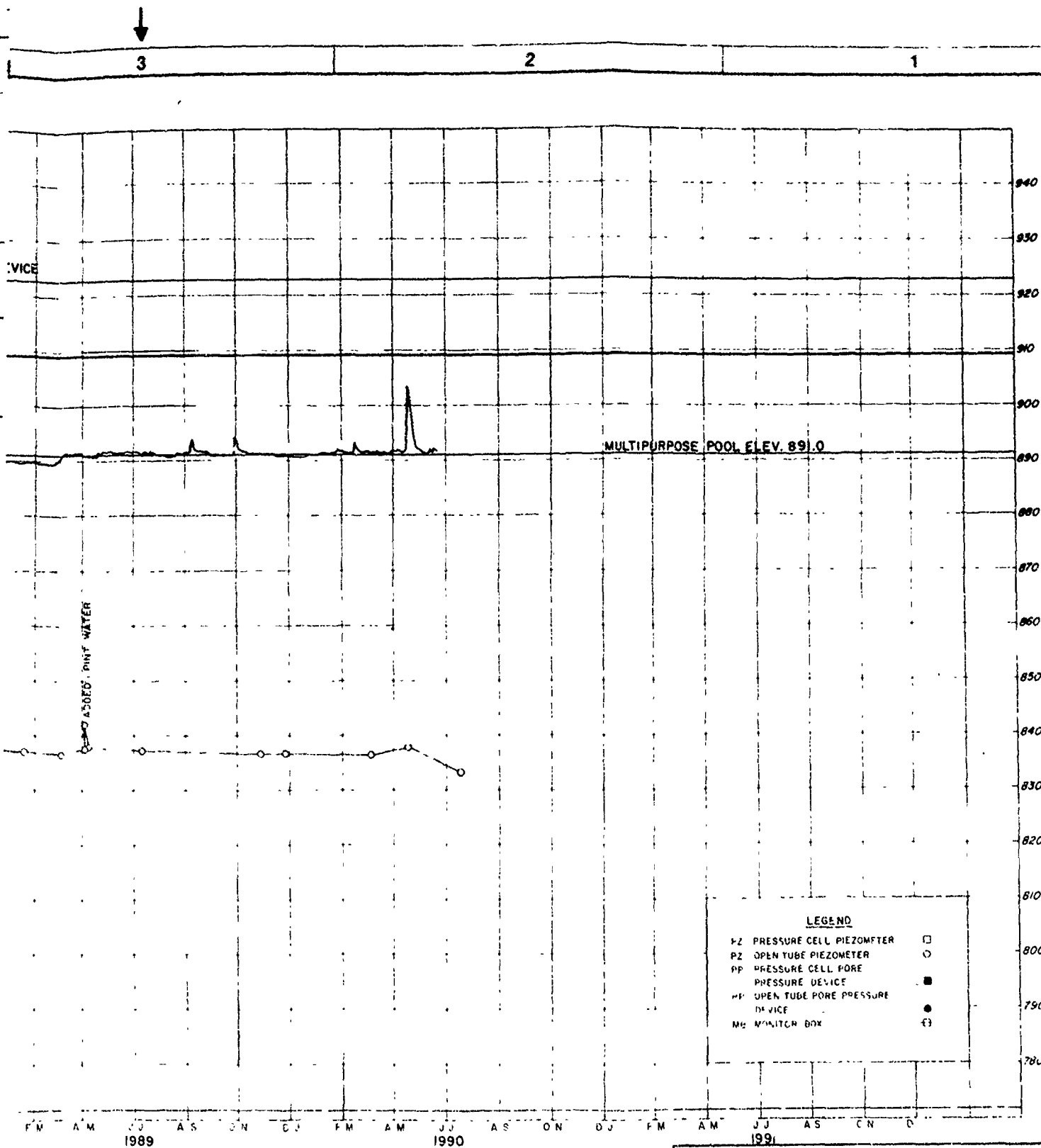












Revisions		Date	Approved
Symbol	Descriptions		
	Revised March 1990		

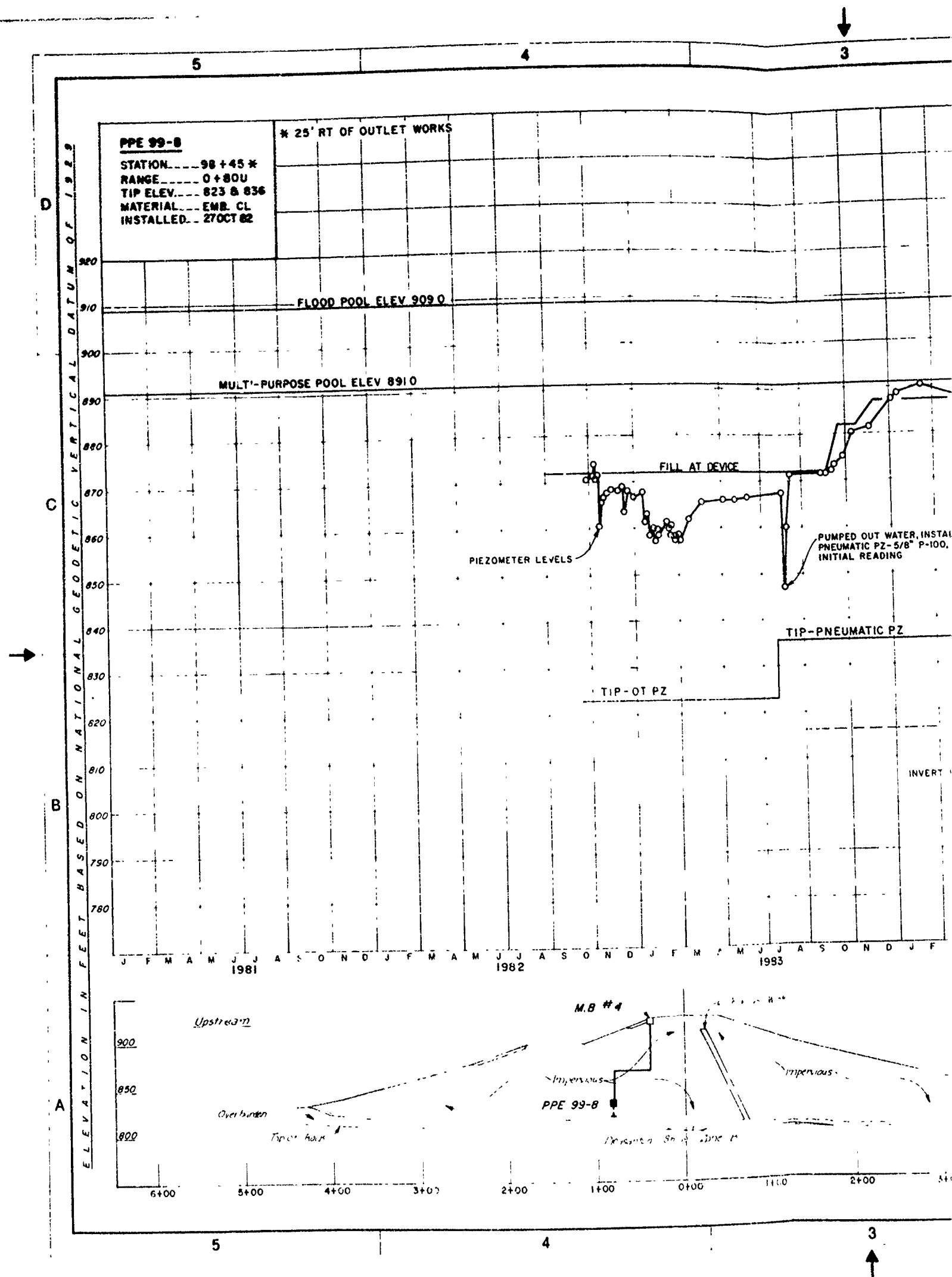
U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: RGD  
DRAWN BY: ECR  
CHECKED BY: RLA  
SUBMITTED BY: WHE

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT

PIEZOMETER PLOTS  
OPEN TUBE PZ  
PPF 99-4

AS SHOWN  
DECEMBER 1986  
2 of 2  
RBI-3-1680



3

2

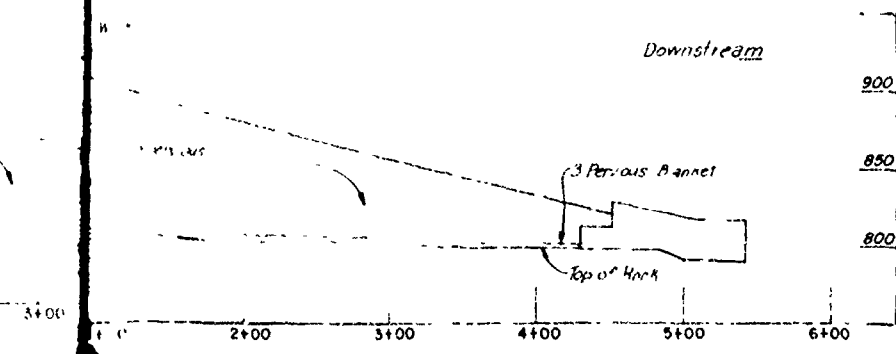
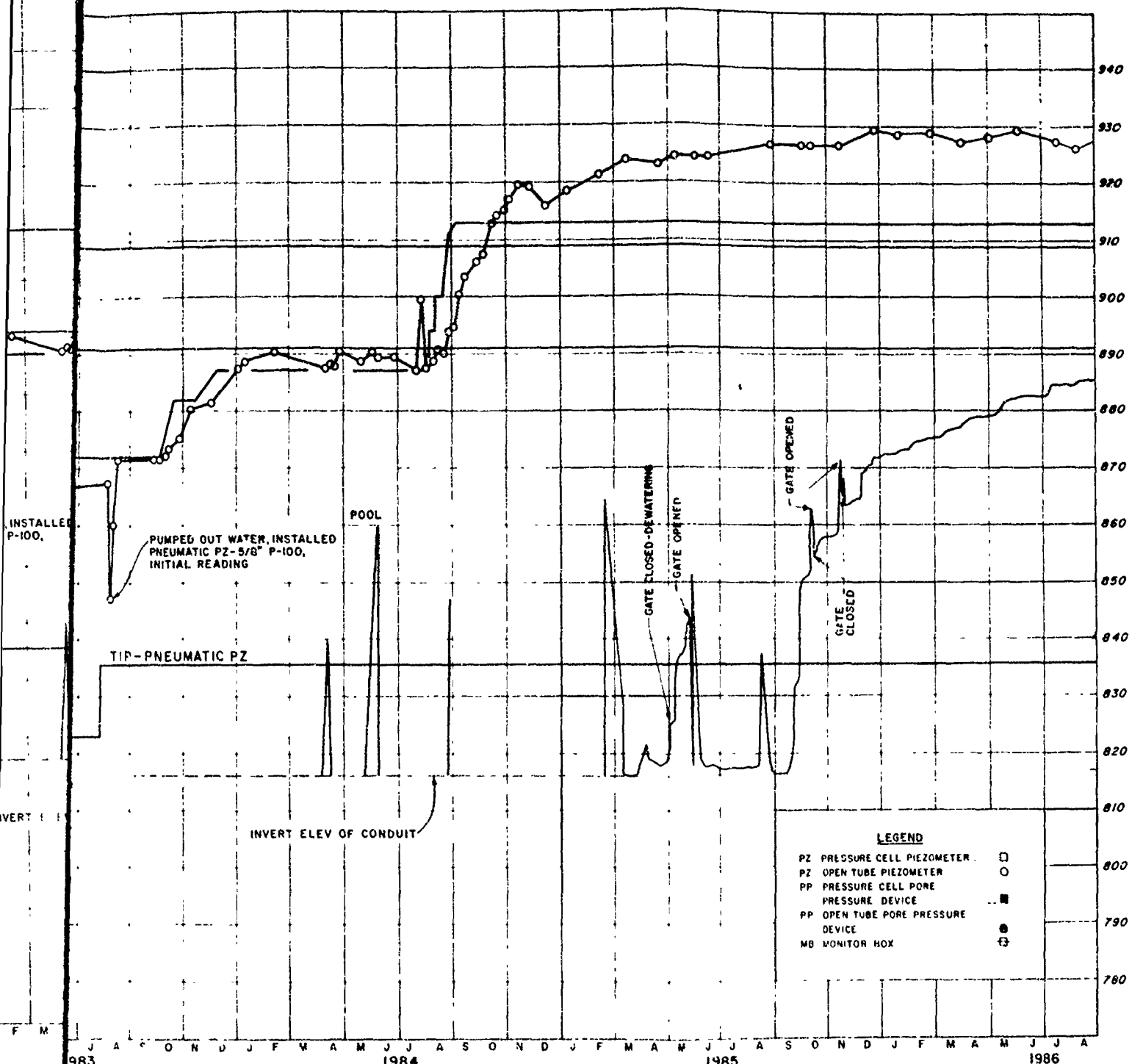
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Revisions			
Symbol	Description	Date	Approved

U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

DESIGNED BY: RGD  
DRAWN BY: EER  
CHECKED BY: RLA  
SUBMITTED BY: WBE

Scale: AS SHOWN  
Date: DECEMBER 1985  
Dwg No: 1 of 2

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA AND  
PERFORMANCE REPORT  
PIEZOMETER PLOTS  
PPE 99-8

Sheet number:  

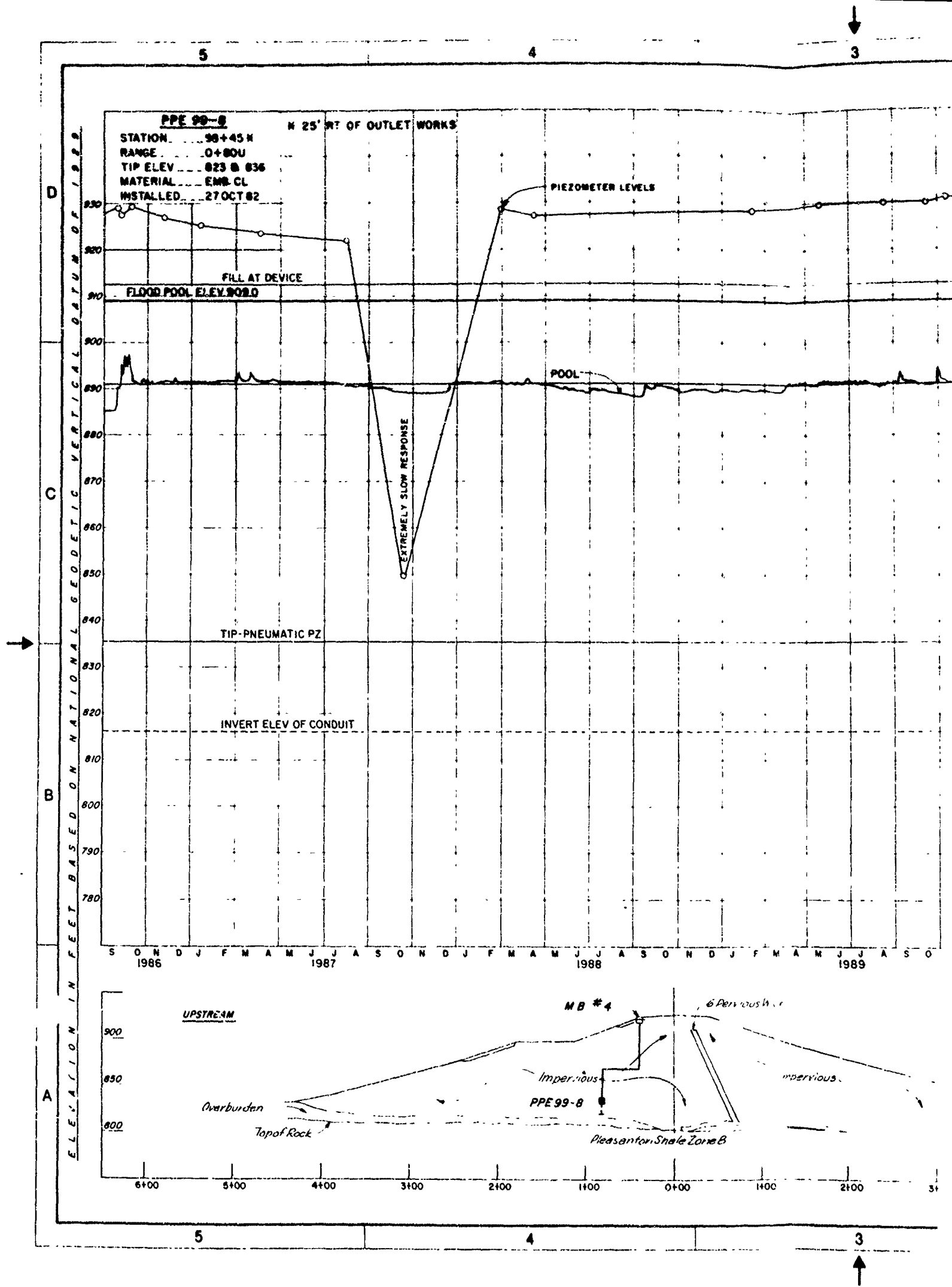
File No: RBL-3-1681

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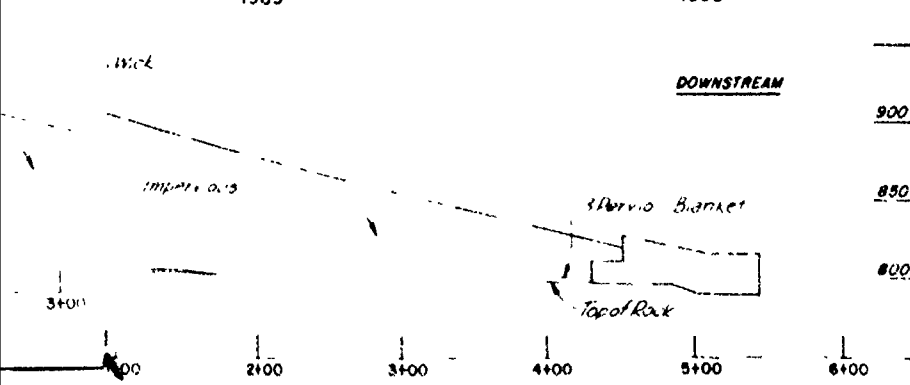
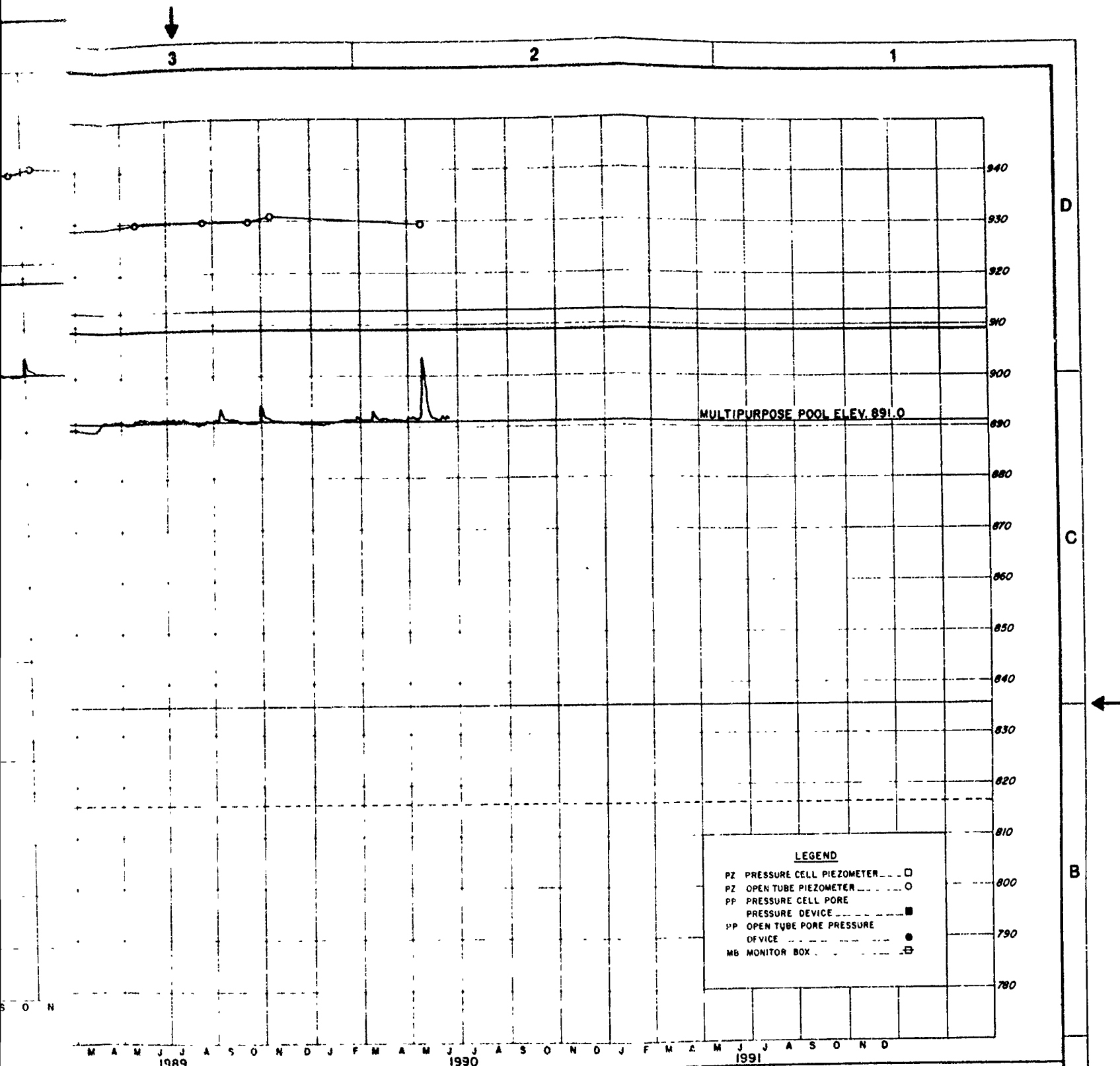
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PLATE NO. 116



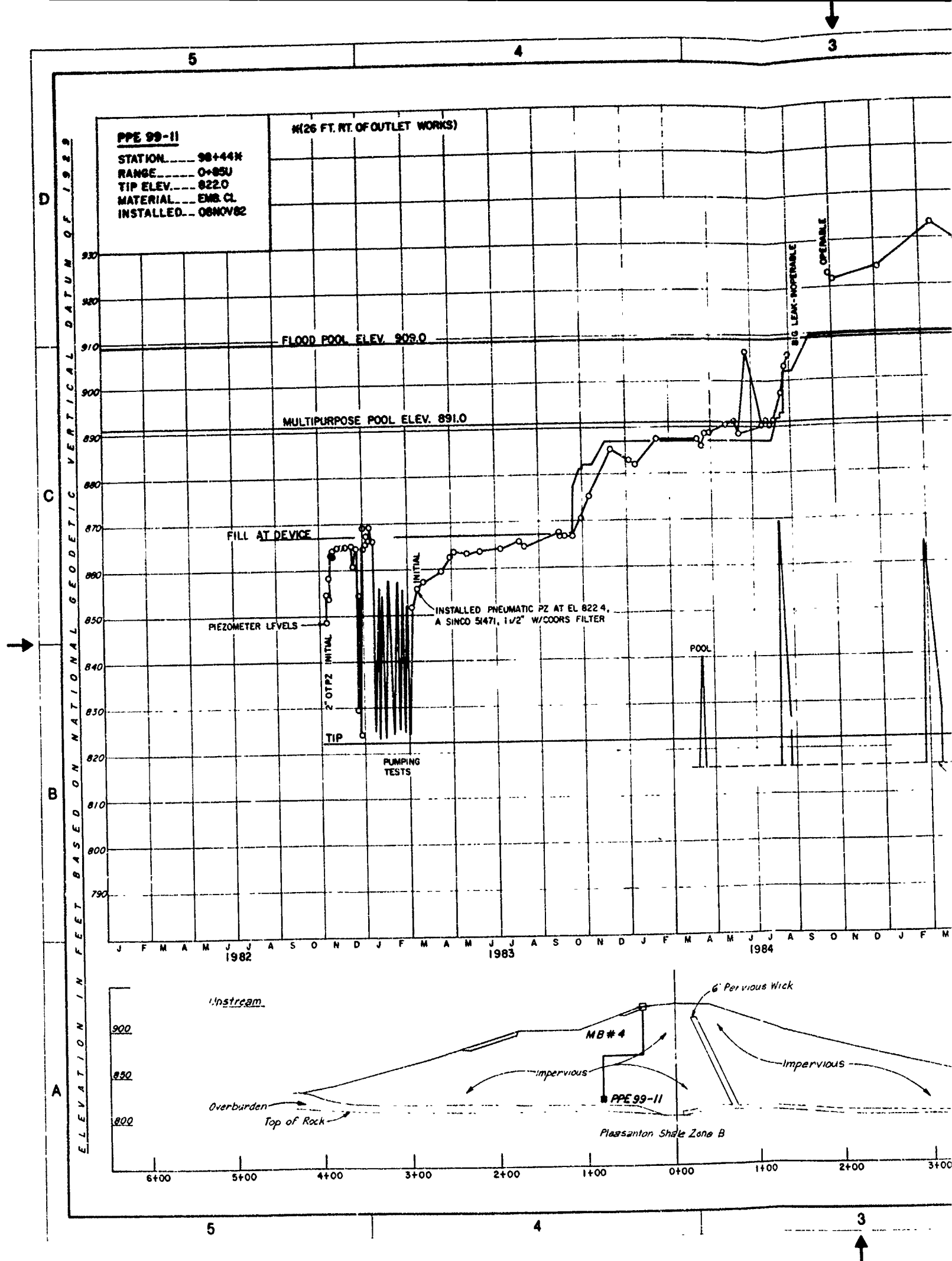


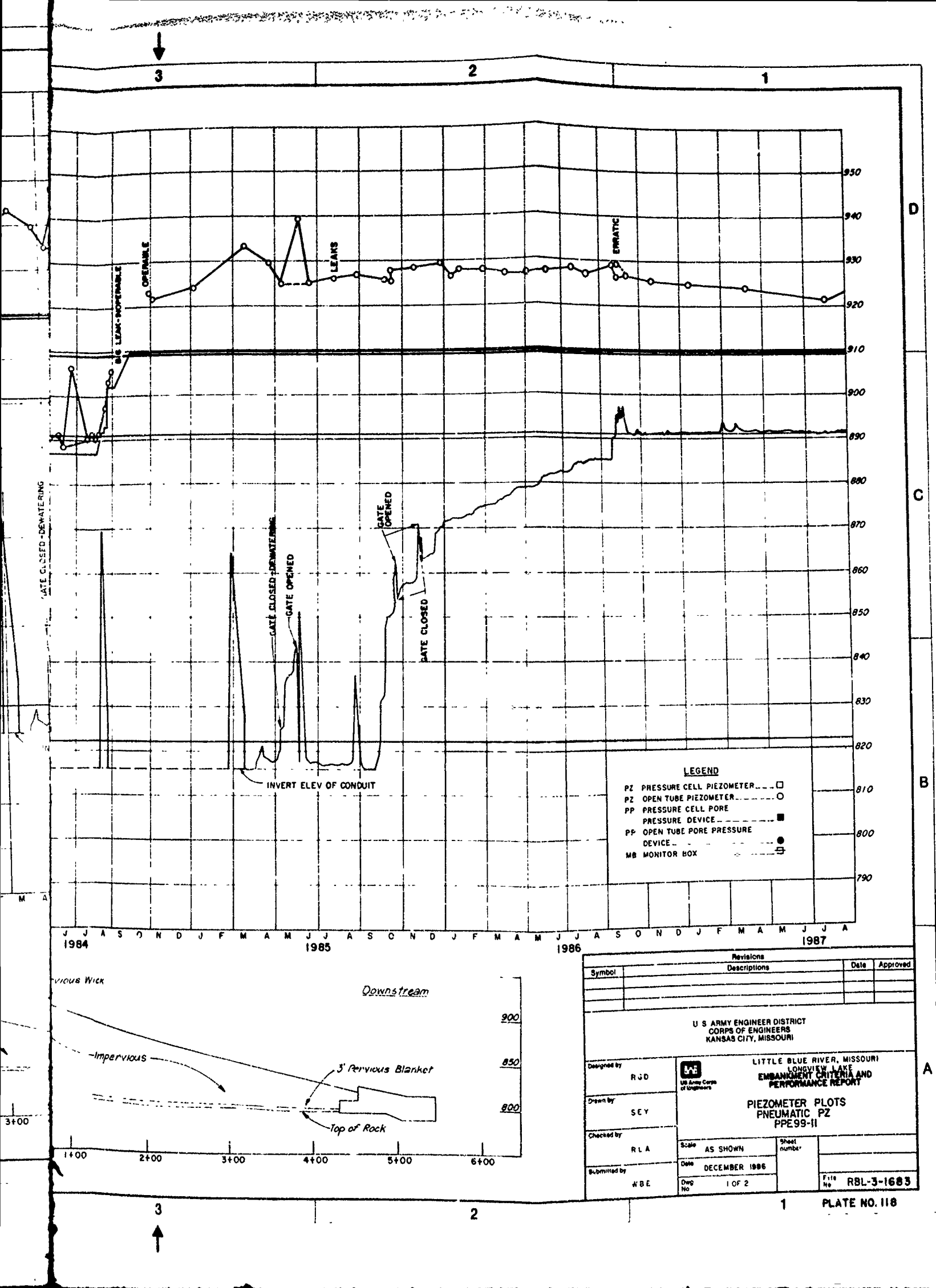


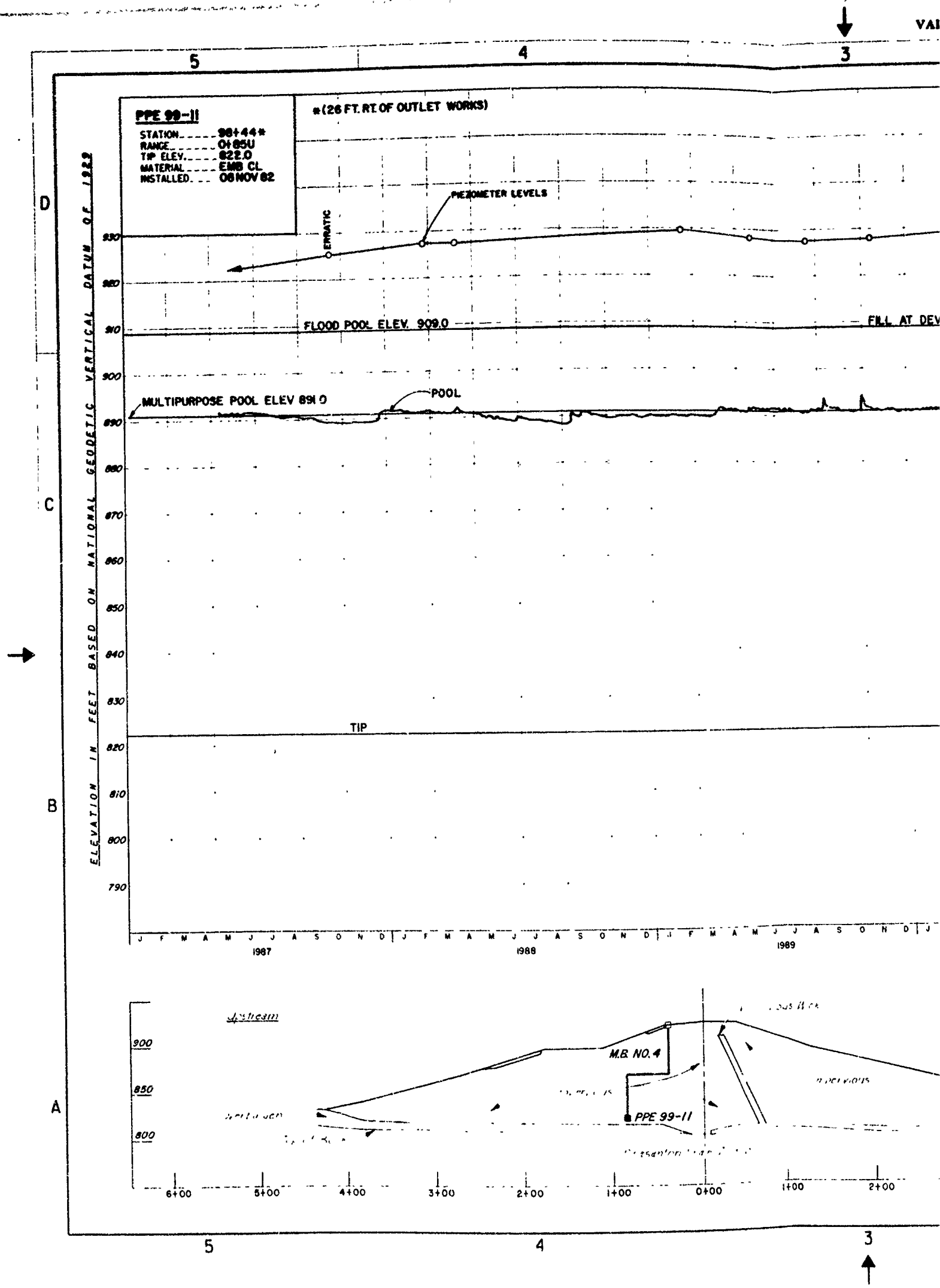
Revisions			
Symbol	Descriptions	Date	Approved
	Revised March 1990		

U. S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Design: T by	RGD		<p>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT</p> <p><b>PIEZOMETER PLOTS PNEUMATIC PZ PPE 99-8</b></p>		
Drawn by	FER				
Checked by	R. L. A.				
Submitted by	W. B. I.				
		Scale	AS SHOWN	Sheet Number	
		Date	DECEMBER 1986		
		Dwg No.	2 of 2	File No.	RBL-3-1682







VALUE

VALUE ENGINEERING PAYS

3

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1

DEVICE

FALL AT DEVICE

PLUGGED

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A

LEGEND

- PZ PRESSURE CELL PIEZOMETER
- PZ OPEN TUBE PIEZOMETER
- PP PRESSURE CELL PORE PRESSURE DEVICE
- PP OPEN TUBE PORE PRESSURE DEVICE
- MB MONITOR BOX

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900

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870

860

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840

830

820

810

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790

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1989

1990

1991

200

850

800

1+00

2+00

3+00

4+00

5+00

6+00

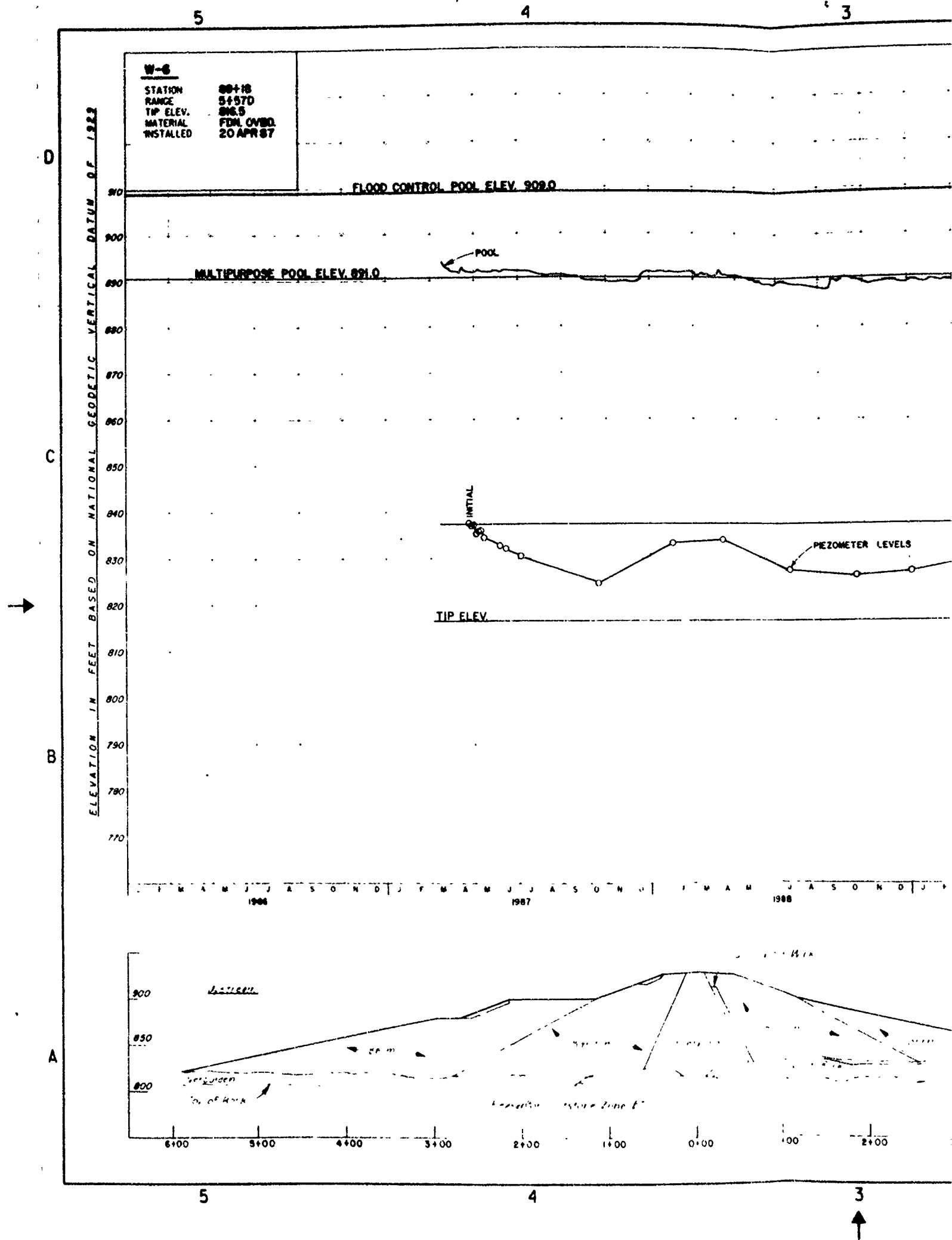
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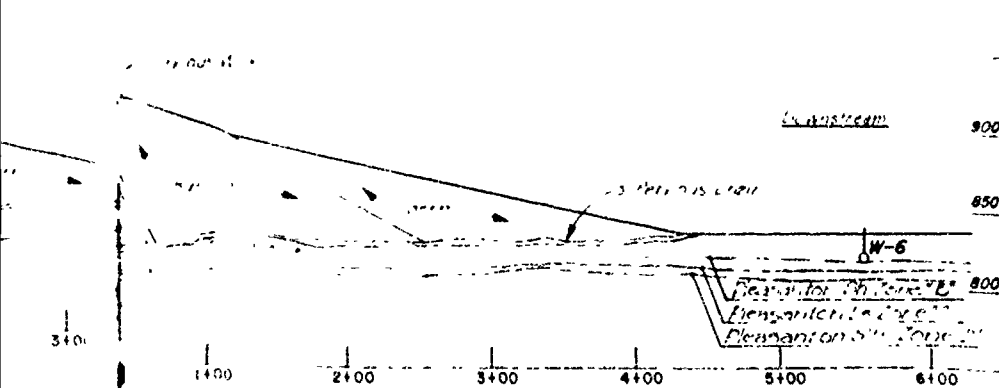
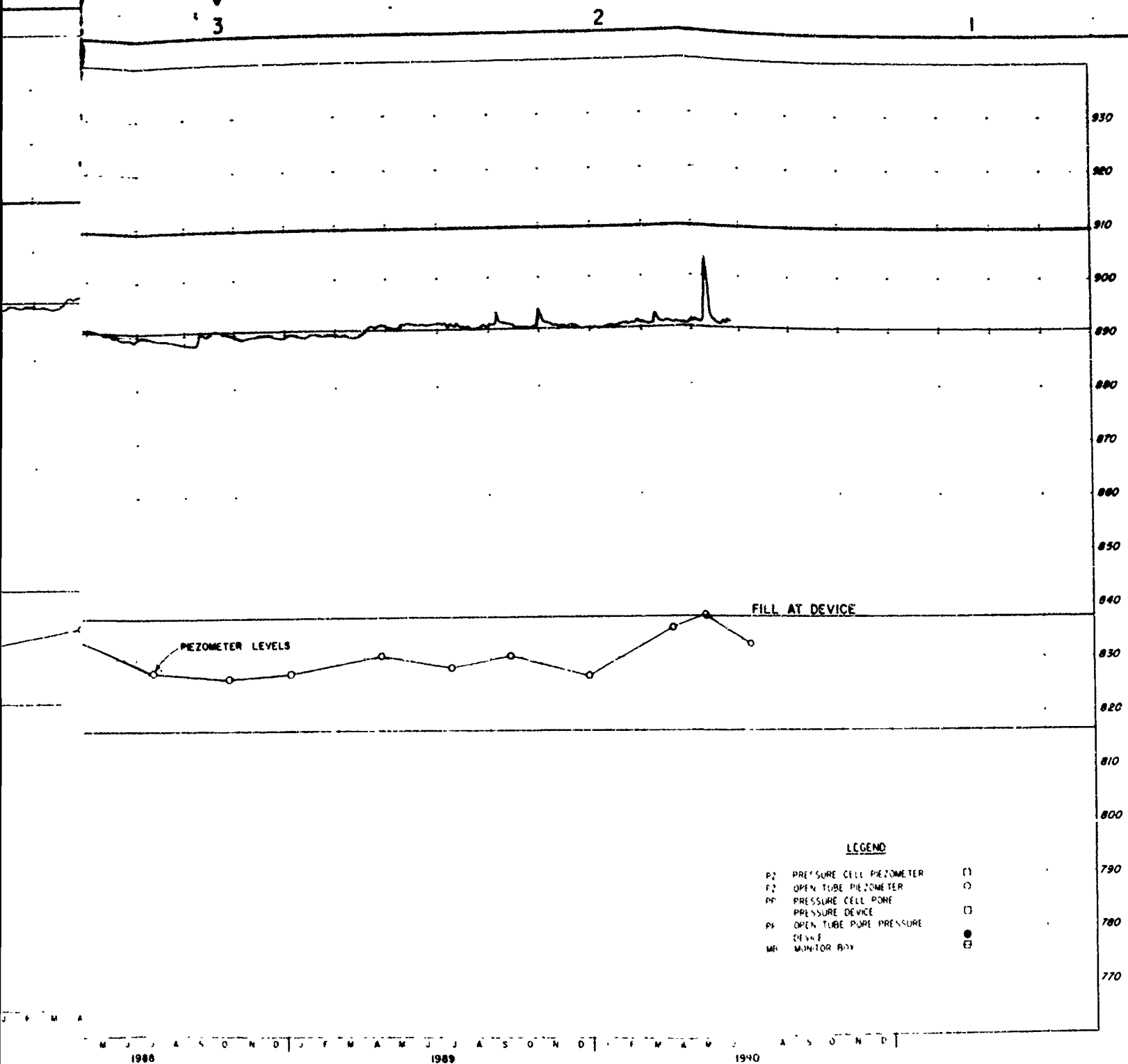
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PLATE NO. 119

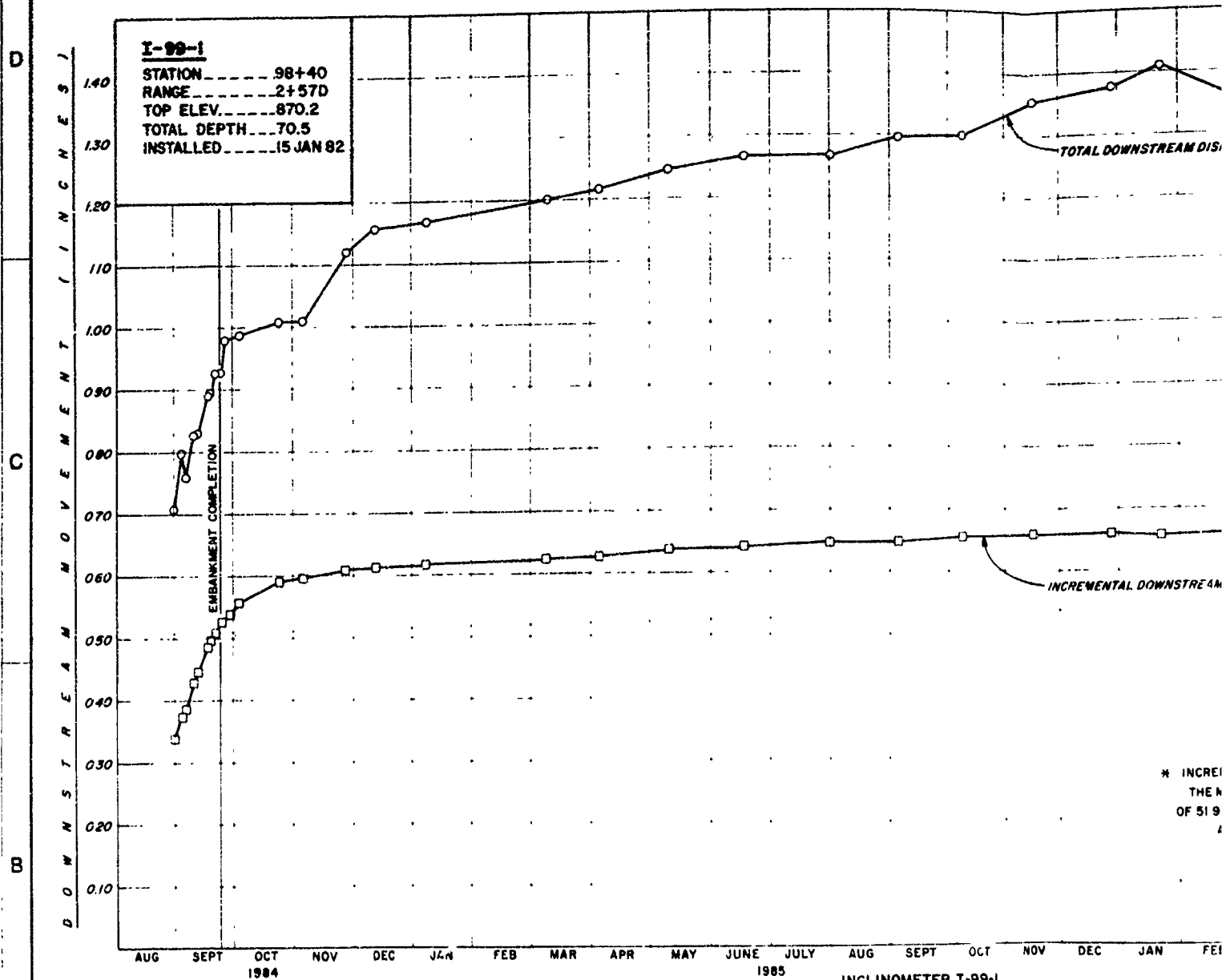
Revisions		Date	Approved
Description			
Revised March 1990			
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI			
LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT			
PIEZOMETER PLOTS PNEUMATIC PZ PPE 99-II			
DECEMBER 98		1100.651 PROFILE RBL-3-1684	



VALUE ENGINEERING PAYS

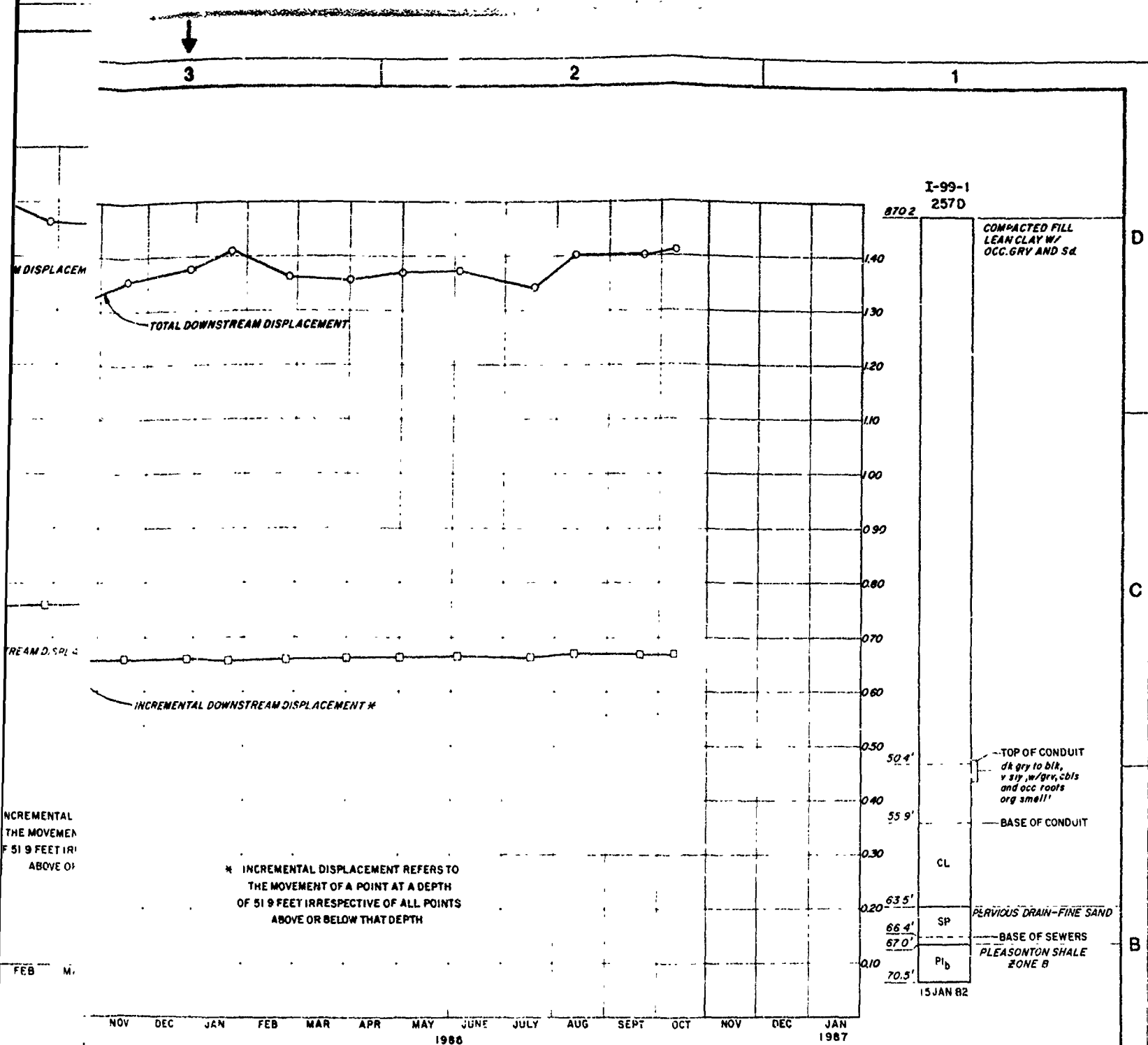


Revised March 1990		Date Approved
U.S. ARMY ENGINEER DISTRICT CORPS OF ENGINEERS KANSAS CITY, MISSOURI		
LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT		Sheet Number
PIEZOMETER PLOTS OPEN TUBE PZ W-6		
Author JFM	Check JFM	Post Code 1211
Date DECEMBER 1986	Scale AS SHOWN	Design File RBL-3-1685



INCLINOMETER I-99-1  
INCREMENTAL DOWNSTREAM DISPLACEMENT VS. TIME  
DEPTH: 51.9 FT. ELEVATION: 818.3





**DISPLACEMENT VS. TIME**  
**I: 818.3**

Revisions			
Symbol	Descriptions	Date	Approved

**U S ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI**

Designed by <b>RGD</b>	<p><b>LITTLE BLUE RIVER, MISSOURI LONGVIEW LAKE EMBANKMENT CRITERIA AND PERFORMANCE REPORT INCLINOMETER DISPLACEMENTS I-99-1</b></p>	Scale <b>A3 SHOWN</b>	Sheet number
Drawn by <b>DEJ</b>		Date <b>DECEMBER 1986</b>	
Checked by <b>RLA</b>			
Submitted by <b>WBE</b>			

File No. **RBL-3-1697**

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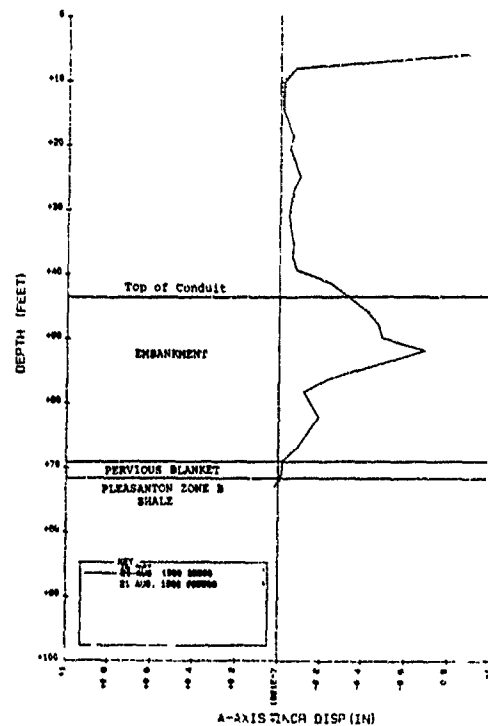
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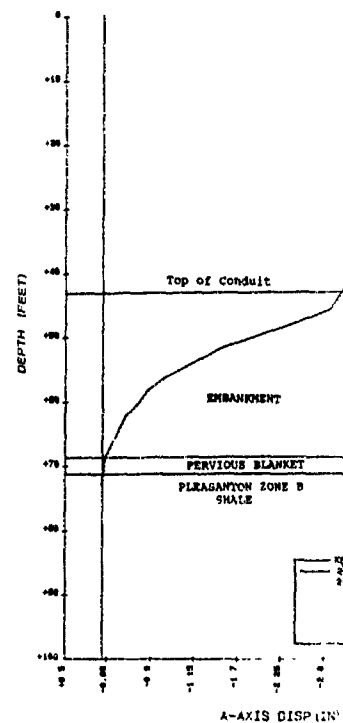
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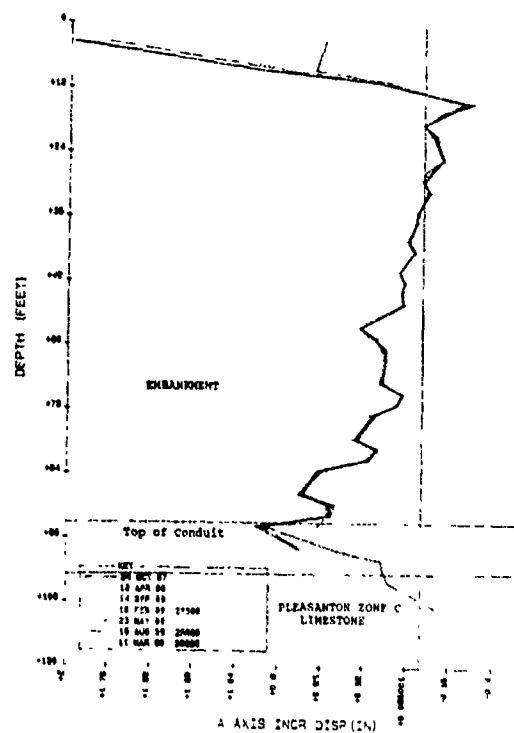
LONGVIEW LAKE I-99-1 R2+570



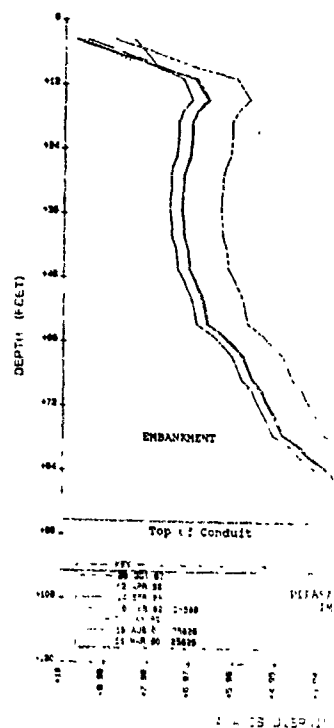
LONGVIEW LAKE I-99-1 R2+570



LONGVIEW LAKE I-99-2 R0+85U



LONGVIEW LAKE I-99-2 R0+85U



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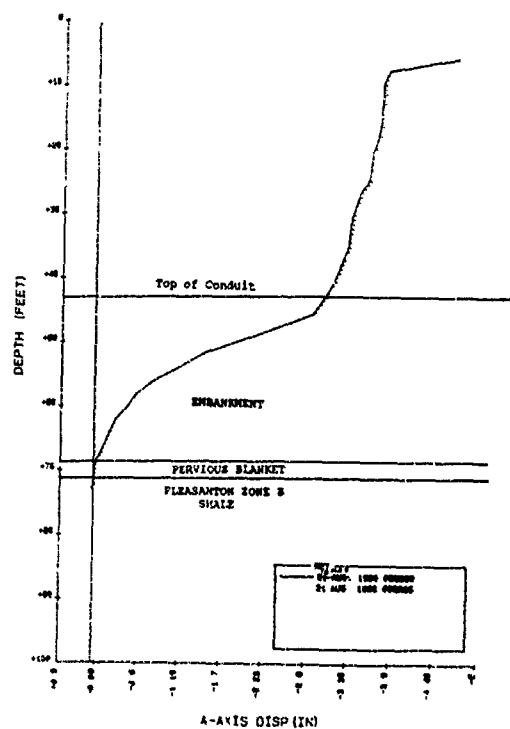
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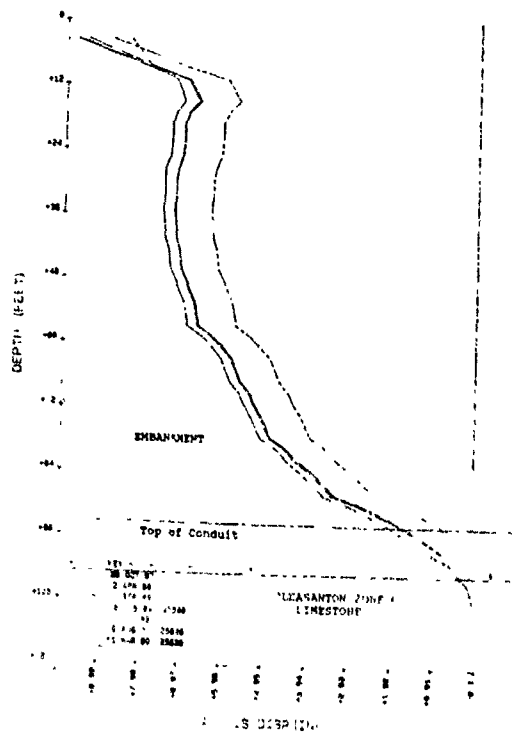
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LONGVIEW LAKE I-99-1 R2+570



LONGVIEW LAKE I-99-2 R0+850



Revisions			
Symbol	Descriptions	Date	Approved

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

Designed by: WBE  
Drawn by: KOS  
Checked by: LLL  
Submitted by: RDC

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

**INCLINOMETER DATA PLOTS  
I-99-1 AND I-99-2**

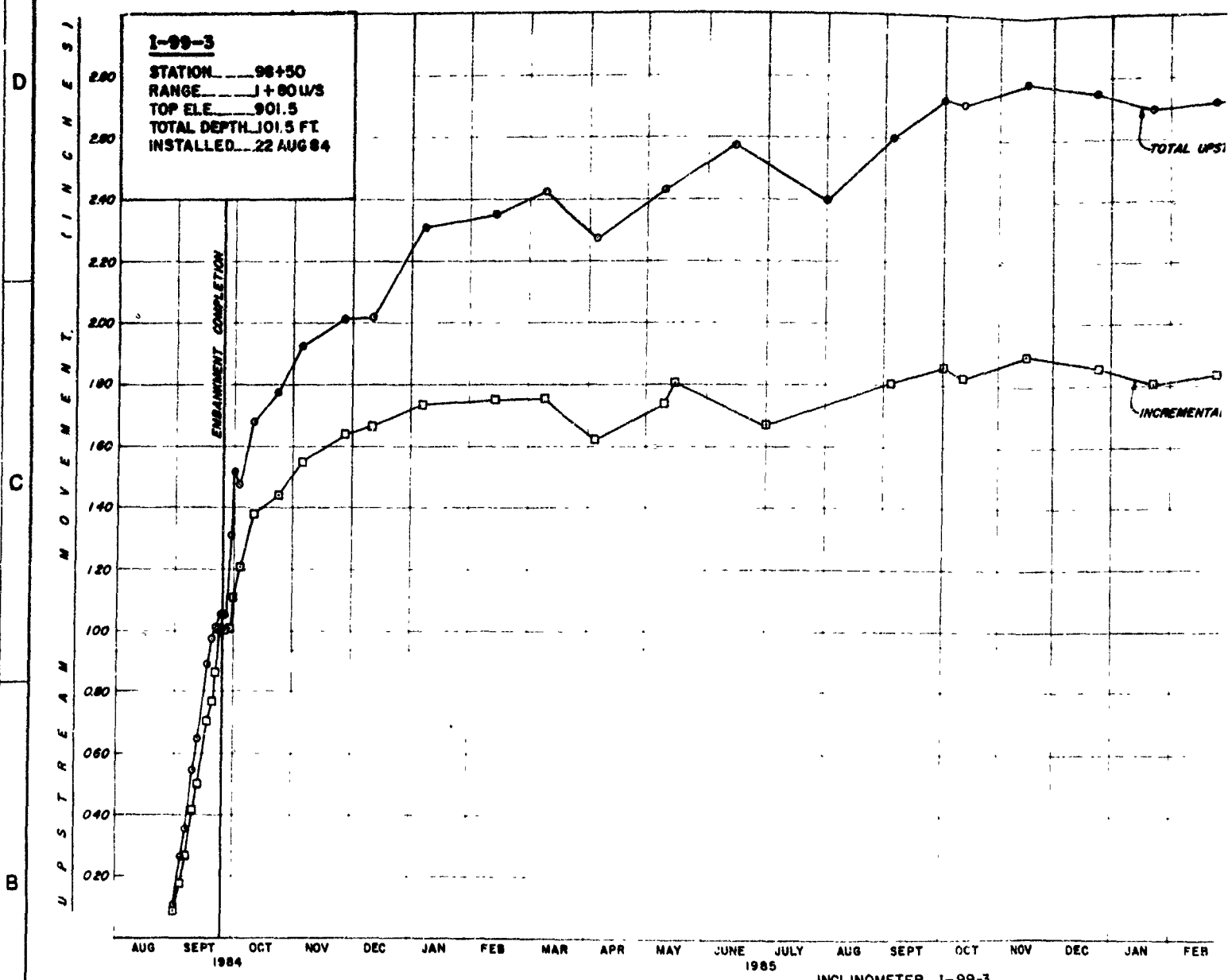
Scale: AS SHOWN	Sheet number:	Plot Scale:
Date: JANUARY 1991		Design File:
Dwg. No.:		File No.: RBL-3-1698

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PLATE NO. 122



INCLINOMETER I-99-3  
TOTAL AND INCREMENTAL UPSTREAM DISPLACEMENT VS TIME  
DEPTH 74.5' ELEVATION 827.0

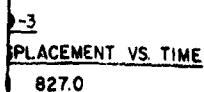
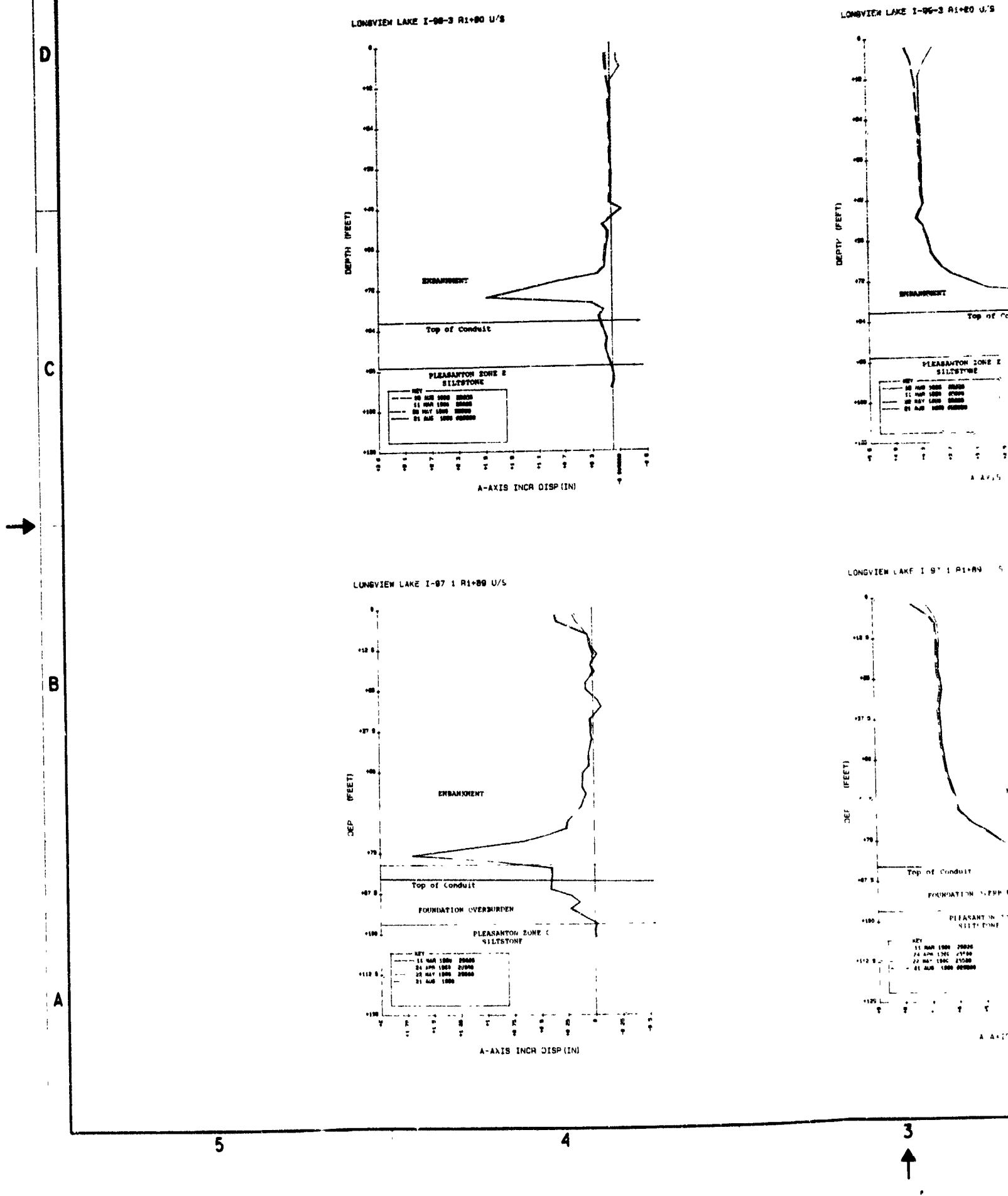


PLATE NO. 123



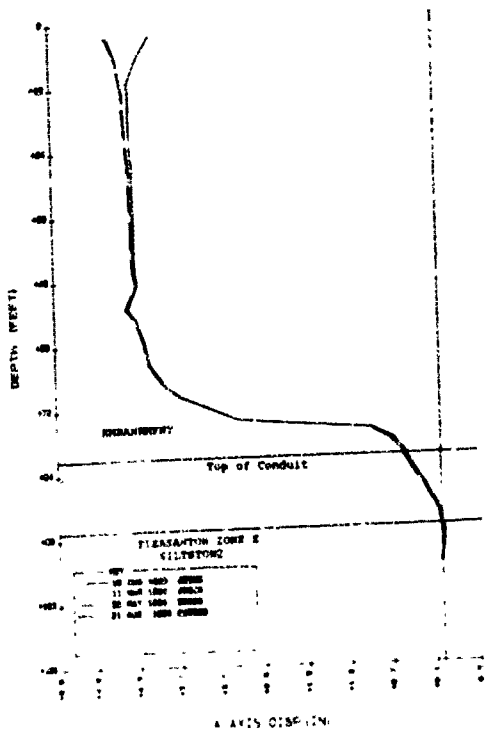
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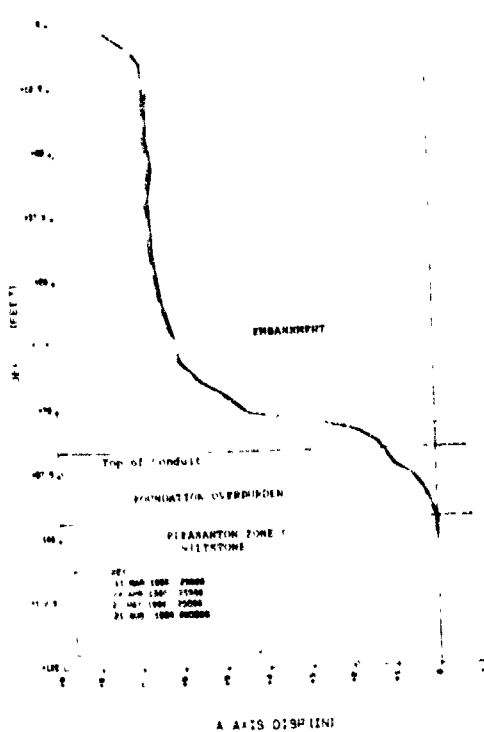
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LONGVIEW LAKE 1-99-3 R1-80 U.S.



LONGVIEW LAKE 1-99-3 R1-80 U.S.



Symbol	Revisions	Descriptions	Date	Approved

U.S. ARMY ENGINEER DISTRICT  
CORPS OF ENGINEERS  
KANSAS CITY, MISSOURI

LITTLE BLUE RIVER, MISSOURI  
LONGVIEW LAKE  
EMBANKMENT CRITERIA  
AND PERFORMANCE REPORT

INCLINOMETER DATA PLOTS  
1-99-3 AND 1-97-1

Designed by: WBE  
Drawn by: ADS  
Checked by: LLS  
Submitted by: RDU

AS SHOWN  
JANUARY 1998

RBL-3-1700